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Engineering News-Record

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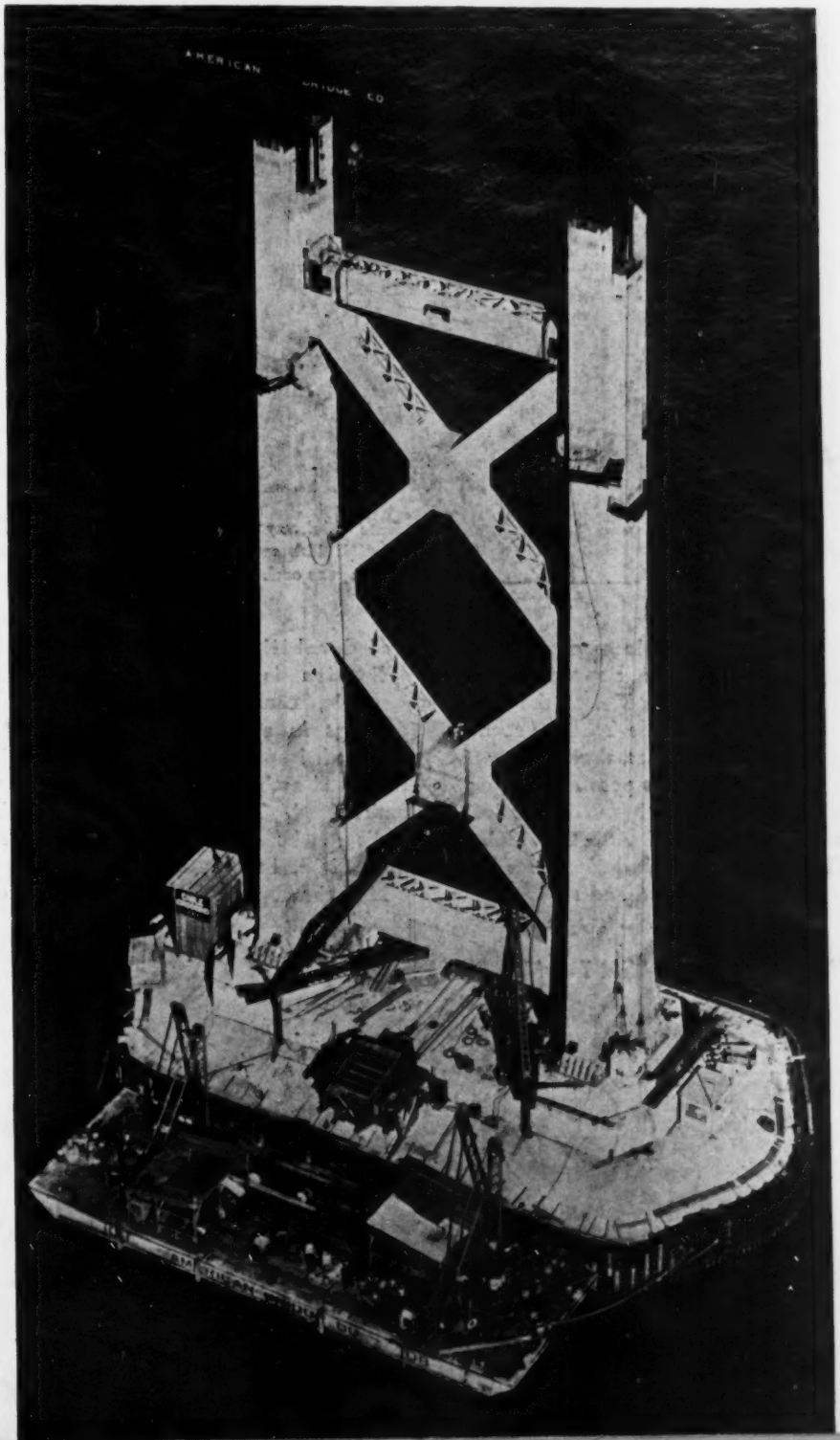


Hammerhead derricks carried in center wells of tower legs erecting one of the towers for the twin suspension spans in the San Francisco-Oakland Bridge.

Bay Bridge Tower Design and Erection

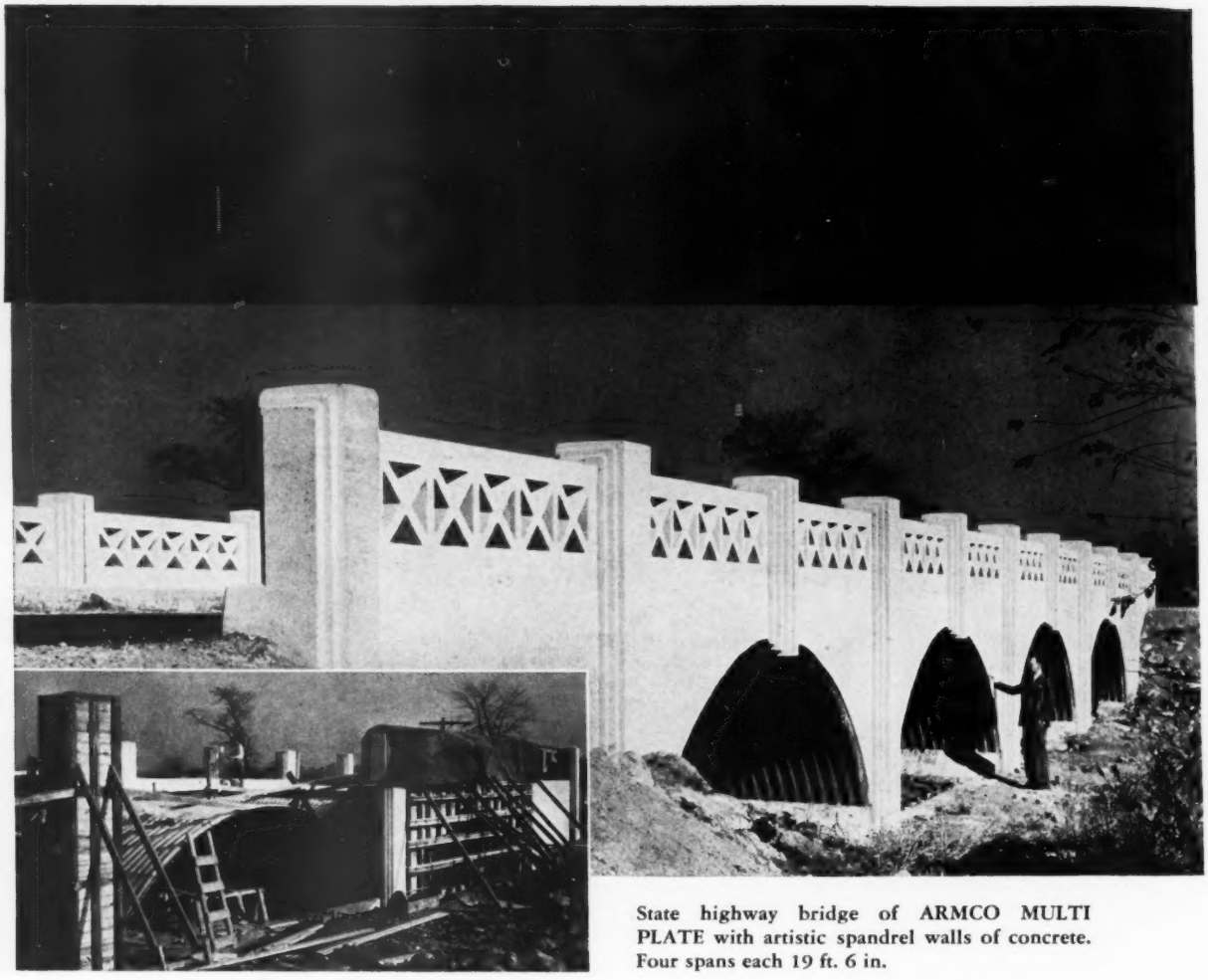
Concrete Roof Arches Span 282 Feet

Timber Floor Design for Earthquakes

Driving Sewer Tunnels in Blue Clay

Snow Surveys Aid Flood Forecasting

Soil Types and Optimum Moisture



State highway bridge of ARMCO MULTI PLATE with artistic spandrel walls of concrete. Four spans each 19 ft. 6 in.

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Don't Hamstring the Highway Program

IN THE June issue of *Construction Methods* appears this photograph of Korean coolies on a construction job. It is significant that an American construction journal should assign such a picture to its "Job Oddities" department, devoted to items that are arresting or unusual.

Gates Photo

Unusual, that is, until the advent of the Works Progress Administration, under its present leadership; for recent developments suggest that that administration might appropriately carry this picture on its letterhead as expressing its guiding spirit. When the administration requires that work-relief projects must be limited to those on which no more than $1100 to $1400 per man per year may be spent for labor, materials and incidentals, it prescribes in effect that the works shall be conducted in exactly the fashion here illustrated.

We are told that these unit figures were derived by the simple process of dividing $4,000,000,000 by 3,500,000 workers. If the unemployment problem can be solved so patly by so simple an arithmetical exercise, it seems too bad that we did not dispose of it two years or more ago. It is evident, however, that it does *not* solve that problem, that it does not even help to solve it and that, moreover, as it now is interpreted and applied, it may contribute substantially to confound the President's efforts.

BE IT noted that such a computation assumes the employment of 3½ million men under the government work-relief program for a year. But is it not the purpose of the program to reduce the number of unemployed? If the program is successful, must not this 3½ million constantly diminish? Does not this calculation rather take for granted the failure of the President's program, with its assumption that we shall have 3½ million unemployed throughout the coming year? And what then? Are we to expect another $4 billion appropriation to make employment for the same 3½ million during another year?

Quite to the contrary, the President's program contemplates a steady reduction in this roll of unemployed. And those responsible for its administration should see to it that every dollar is so expended as to effect such a reduction, not to insure its perpetuation. But such a reduction can be realized only through the absorption of the unemployed into normal occupations, some of them in regular governmental operations but most of them in private business. The general recovery of private business will depend, in the long run, on the operation of certain economic processes and upon those policies of government that affect them. On them the work-relief program can have little direct effect.

It can, however, and does have a profound effect on that important group of industries which at all times and especially at present must find a major market in governmental use. Sound work-relief administration will insure that while it is creating the maximum of emergency employment, it does not kill existing normal employment and obstruct its present growth. In commenting on the President's program as outlined in his January message, this page said: "But every precaution must be exercised to prevent such an emergency program from scuttling the organized construction industry. Construction and its dependent industries are vital factors in the national economy; it would be futile to absorb the unemployed of other industries by means that would undermine it." Today this comment still is pertinent.

IT IS quite possible that the Works Administration can justify its methods on much of the emergency work that can be created to provide direct employment. Many communities offer many projects that rationally would require a large proportion of unskilled labor on the job. Resourceful city engineers have done well by their communities in using their unemployed labor and relief funds to do jobs that normally would not have been undertaken. Thus they do not supplant regular employment; their work represents a net employment gain and does not obstruct the progress of private business toward self-support.

But when such a principle is applied recklessly to the road-building program, it breeds more harm than good. It violates several of the basic principles laid down by.

(Continued on next page)

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the President, especially that which he designated as Number 1. It reads as follows: *"All work undertaken should be useful—not just for a day, or a year, but useful in the sense that it affords permanent improvement in living conditions or that it creates future new wealth for the nation."* Farther along in his message the President included specifically as desirable projects "the improvement of existing road systems and the construction of national highways *designed to handle modern traffic* and the elimination of grade-crossings."

EVERY practical man knows that the mere grading and ditching to which road-building is reduced by the restrictions of the Works Administration will not "create future new wealth for the nation." On the contrary, it must be evident to all that it will produce a large mileage of earth grading that would in no sense constitute highways "designed to handle modern traffic." Without surfacing and further improvement such grading would deteriorate until eventually the total expenditure could be charged off to sheer waste.

Turning now from the wastefulness of such a policy, specifically denounced by the President, we find that as a measure of relieving unemployment it is decidedly ill-considered.

During the last two years the highway construction programs have made a distinct contribution toward stimulating employment and fostering natural recovery. They have accomplished this because they have been carried on by normal processes using normal facilities. On the strength of those programs many contractors have recalled men who had been out of a job; manufacturers and other producers have increased their staffs: this the result of road-building operations that, directly and indirectly, have put hundreds of thousands back to work. A continuance of the same rational program would mean further progress toward the end sought by the President. Remember that he has emphasized the need that the program "give employment to all of these 3½ million employable people now on relief, *pending their absorption in a rising tide of private employment.*" The italics are by this writer; they are designed to show that the President definitely has in mind the urgent need of stimulating private employment as the essential part of his program. He did not contemplate stealing the jobs of those now employed and of the many worthy skilled and unskilled workers, now out of work, who are living on relatives or on their own savings, so that he could turn them over to those already on relief rolls. Neither did he contemplate grabbing off enough private industry jobs to maintain a permanent relief roll of 3,500,000 at an annual cost of $4,000,000,000.

WHAT then can be said for a policy that thus contemplates the undermining of normal employment in private enterprise in the name of unemployment relief? How can we justify the disruption of a road-building program that during the last two years has provided many miles of practical, useful, modern highways while it created normal private employment, and the substitution for it of a downright CWA, manual-labor project such as the new program contemplates? Is it not evident that the extension of these principles from the field of "made-work" into that of normal public works construction will but increase unemployment among skilled workers more rapidly than it can provide hand-labor jobs at subsistence wages? How can that make for either relief or recovery?

That this is no academic question already is indicated by developments in those industries most immediately dependent on governmental construction in general and road-building in particular. One manufacturer reports that "in our own plant we have arranged to go into production of numerous items which we make and have been holding off because we suspected a botching of this highway program. If this work were going to be carried through normally," he continues, "we would need to recall a great number of our skilled workers, engineers, draftsmen, etc., as well as a great number of our unskilled workers." This is but typical of hundreds of plants in scores of industries, who gradually have been stepping up their operations with the progress of recovery and who now see in this sabotage of normal practice a threat sufficient to give them pause.

Much more might be said concerning this failure of the Works Administration to sense the damage it is doing to the cause for which it was set up. Its ill-considered reliance on an arithmetical ratio to solve so complicated a problem produces many inconsistencies which, in the aggregate, will defeat its purpose. To cite but one: highways designed to handle modern traffic, as the President put it, in the neighborhood of large industrial centers and grade-crossing eliminations, which are most needed in the same areas, cannot be built under the terms of these regulations; and it is in just these areas that we find the heaviest unemployment. Obviously here is a situation that violates the President's principles with no excuse other than devotion to a rule-of-thumb answer to a complex, far-reaching industrial problem.

IT HAS been well suggested that state highway departments should use their work-relief appropriations exclusively to pay for ditching and grading on highways that can be further improved through Federal Aid funds matched by their states. Obviously this may offer some measure of offset to the unfortunate restrictions of the Work Relief Administration. Every engineer charged with responsibility for these projects should do as little as possible of his grading and drainage with his normal highway funds and as much as possible with the work-relief funds; thus more of the former will be available for turning otherwise futile grading into practical, useful and efficient highways for his state.

It must be evident, however, that the most careful observance of such a policy cannot meet the needs of the situation in every state. While this should be done wherever practicable, the real solution must be sought in a revision of the announced program. Whether this be accomplished by increasing the allowance per man-year for highway work to a figure more nearly consistent with the requirements of practical highway construction, or by including in the appropriation estimates the indirect as well as the direct labor on the job is open to consideration. The fact remains, however, that unless the program is revised we shall have our road-building activities, already a substantial influence for unemployment relief and industrial recovery, hamstrung by ill-considered and impractical restrictions.

Let us use our road-building as a means to conserve jobs, not to destroy them; to protect the jobs of worthy workers, not to steal them. Let us, in doing this, follow the President's counsel and build useful highways, designed to handle modern traffic rather than squander the highway appropriations in aimless grading operations. Useful works, normal employment, quick results: these are the announced objectives of the President for his work-relief program. He can realize them substantially through a road-building program if he will but keep Korean construction policies out of the American recovery effort.

Willard Chevalier

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In the News:

WORK RELIEF remains in the news and up in the air, although the Hopkins plan to limit expenditures to an average of $1,100 per man-year will apparently stick. Mr. Hopkins believes that the average may be maintained by including some higher-cost projects to be balanced by some exceptionally low-cost projects. He also thinks the $1,100 expenditure will create employment not only for one field man, but for one man "behind the lines." State works progress administrators assembled in Washington on Monday and Tuesday for instructions.

NRA, SAVED FROM EXTINCTION by Congress but restricted in scope by the Supreme Court, will continue as a statistical and fact-finding body until April, 1936. The President has appointed James L. O'Neill, of the Guaranty Trust Co., New York, acting administrator.

PWA CELEBRATED its second anniversary this week with some statements of accomplishment. Some 2½ billions of its 3½-billion dollar fund have been spent. Under way or completed are 1,087 water projects, including 744 complete new systems; 622 sewer and sewage-plant projects costing about $200,000,000; 250 bridges, tunnels, subways and "engineering structures" costing over $200,000,000; and more than 35,000 miles of roads and city streets involving an expenditure of $579,000,000. Some $550,000,000 was loaned on non-federal projects.

AN ATTEMPT to set up a licensing system for contractors in New York State is being revived now that codes are no longer operative. Those favoring such a plan think it will keep out the "shoestring" contractors. Opponents think it will turn into a bureaucratic political set-up that can easily be circumvented.

BETTER HOUSING DAY, June 15, sponsored by the Federal Housing Administration, witnessed the start of construction of 3,500 model residences in all parts of the country.

EXPANSION OF THE CCC to a new peak strength of 600,000 men began June 15. This means the employment of about 250,000 additional men.

THE "WISCONSIN PLAN," to set up a finance authority to administer all public works and work relief in the state, free from federal restrictions, was defeated by the state senate 19 to 14. The federal government had allotted $100,000,000 to the state on a 70-30 loan grant basis subject to the state raising an additional $109,000,000.

In This Issue:

THE TOWERS of the twin suspension bridges that comprise the West Bay crossing of the San Francisco-Oakland Bridge are the most recent element of this great project to be completed. Their design, to meet requirements of exceptional lateral stability, and their erection by a wholly new procedure, involving the use of hammerhead derricks, are presented in the two lead articles of this issue.

THE SECOND OF THE THREE ARTICLES to be published on Chicago's 27-mile sewer-tunnel system covers driving methods on bores ranging in sizes from 17½x19 ft. down to such small sizes that Shetland ponies have to be used on the muck cars. Mostly the material encountered is typical Chicago blue clay.

FOLLOWING THE WIDE DESTRUCTION of masonry-wall buildings with timber floors in the Southern California earthquake of March 10, 1933, clay products and timber interests instituted a comprehensive series of full-scale and model tests of floors and wall-floor anchorages, to develop, if possible, designs that would adequately resist earthquake forces. Out of this study, results of which were presented last year in *Engineering News-Record*, has evolved a design procedure that should be of wide interest. The engineeers who conducted the tests present it in this issue as the conclusion of their work.

ADDITIONAL DATA on optimum soil moisture, as related principally to compaction for earth-dam purposes, is presented, with comparisons given for four natural and four mixed soils.

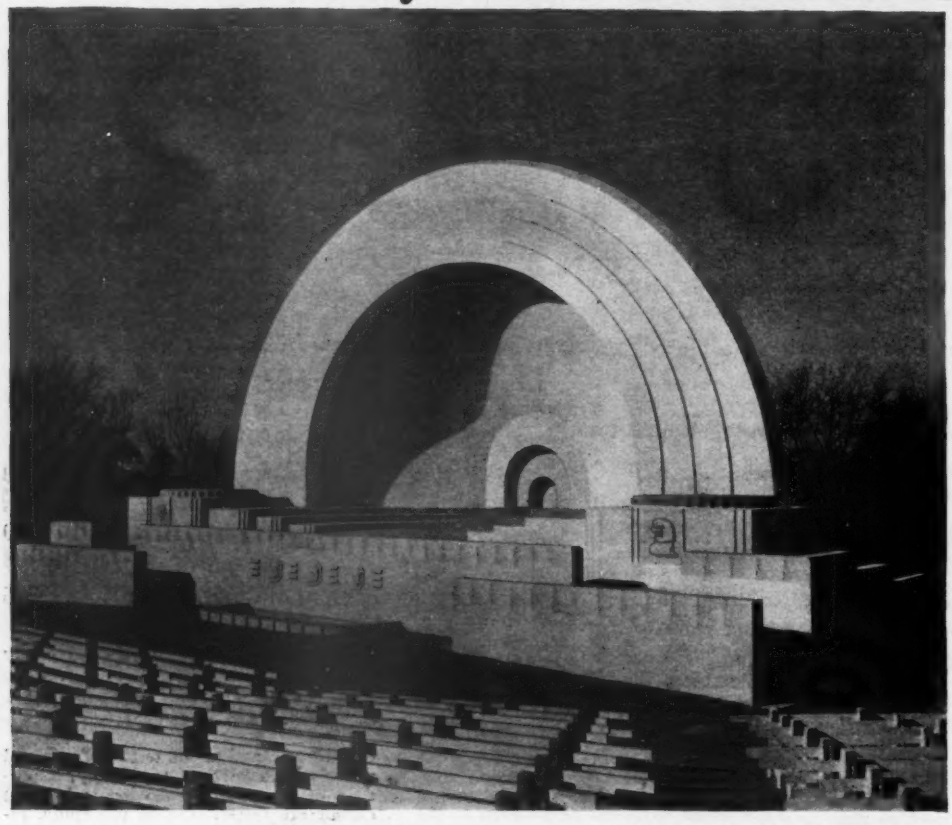
CONCRETE ROOF ARCHES spanning 282 ft. are an outstanding feature of the main building of the current international exposition in Brussels, Belgium. Of three-hinged design, the arches have base hinges of ball-and-socket type.

SNOW SURVEYS as an aid to flood forecasting and control are discussed by J. E. Church, meteorologist of the Nevada Agricultural Experiment Station. Experience in the Tahoe Basin is cited.

Coming Articles:

BOTTLENECKS, formed by narrow stretches of road or by narrow bridges, reduce the efficiency of many of the nation's most modern highways. Massachusetts has had particular difficulty on its shore highways north and south of Boston, where two narrow bridges have constituted serious hazards and caused long delays to traffic. Before this summer is over, however, two bascule spans will be opened over the Fore and Saugus Rivers, that will provide 60-ft. roadways and make these shore roads the express routes that they should be. An article covering these significant structures will be published soon.

DURABLE...*finished in* STUCCO



Winters are not mild in Sioux City—and it takes plenty of *hot* summer weather to grow Iowa's famous corn. So this music pavilion has to take plenty of weather. It *will* take it, and come up smiling, because it is built of concrete with a permanent, durable finish of Atlas White stucco.

"THE work . . . gives promise of *great durability*." That's the way Architect Henry L. Kamphoefner describes his monumental concrete music pavilion erected as a CWA project in Sioux City.

There is only three-quarters of an inch of stucco over the concrete on this project—one-half inch of brown coat and one-quarter inch of white stucco made with one part Atlas White portland cement and two parts Platte River sand, floated down with carpet over wood. But that three-quarter inch of stucco is enough to give a permanently beautiful finish to this permanent, beautiful structure.

For detailed information on durable, beautiful Atlas White stucco, applied in *any* color you choose and in *any* texture you like, for new structures and for remodeling work, write Universal Atlas Cement Co. (*United States Steel Corporation Subsidiary*), 208 South LaSalle Street, Chicago.

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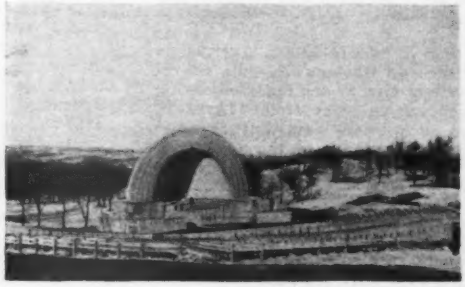
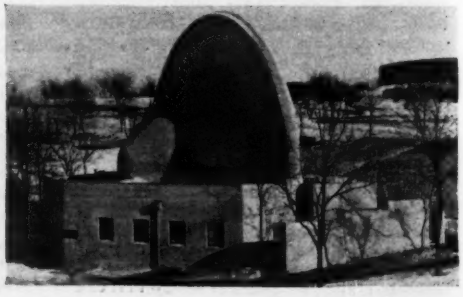
STUCCO *made with* ATLAS WHITE
PORTLAND CEMENT

"Because of the winter dampness on the interior of the building," says Architect Kamphoefner, "the six dressing rooms and toilet rooms were plastered the same as the exterior." That means with durable, beautiful Atlas White portland cement stucco.





ENGINEERING NEWS-RECORD

June 20, 1935

Batter-Leg Towers for San Francisco Bay Bridge

Four towers of twin suspension spans comprising the West Bay Crossing of the San Francisco-Oakland Bridge are designed for exceptional lateral stability with a seismic stress allowance of 0.1*g*.

By C. H. Purcell, Chas. E. Andrew and Glenn B. Woodruff

Respectively, Chief Engineer, Bridge Engineer and Engineer of Design, San Francisco-Oakland Bay Bridge

FIG. 1—TOWERS of West Bay Crossing ready for cable stringing, which began two weeks ago. Towers are numbered 2, 3, 5 and 6 from the San Francisco end. Steel-erection progress on East Bay Crossing may be seen in the background.

FOR THE FIRST TIME in a major suspension structure, batter-leg towers were used in the West Bay Crossing of the San Francisco-Oakland Bay Bridge. Another unusual feature of the design is the arrangement of the column sections with an open central well, which permitted the use of hammerhead derricks, a new means of tower erection. The proportions of the towers were determined largely by the large deflection, which will amount to nearly 7 ft. at the top of tower 2 under extreme conditions.

The West Bay Crossing is a twin suspension bridge; except for differences at the shore anchorages, it is symmetrical about the central anchorage. Each bridge has a main span of 2,310 ft. and side spans of 1,160 ft., except at the San Francisco end, where the span is 1,171 ft. The roadway (58 ft. wide between curbs, on the upper deck) is on a 7,110-ft. vertical curve, with a center ordinate of 48 ft. between towers 2 and 6. This has the result that towers 3 and 5 are 44 ft. higher than towers 2 and 6. Also, on account of the long backstay at the San Francisco anchorage, tower 2 is subject to greater deflections at its top than are the others, and accordingly somewhat greater sections were required. Aside from these differences, all towers are alike above the roadway. Towers 3 and 5 are duplicates throughout. Towers 2 and 6 are alike below the roadway, except for certain reinforcing plates on 2, mentioned later. The tower legs are cellular in construction, with a maximum of 21 cells near the base and 9 just below the top. The cable saddles are fixed to the tower tops, thus requiring the towers to bend in accommodating the structure to live load and temperature changes.

Towers 3 and 5 have a height of 462.4 ft. from top of masonry, at El. +40, to the center of the cable. For adequate lateral stability, a tower width greater than the 66-ft. cable spacing was desirable. Furthermore, the dimensions required for the tower columns were such that, had these columns been vertical, they would have cut into the roadway clearance. Details of the stiffening truss anchors were simplified by not having them in line with the columns, and this also reduced the moment arm of the stiffening-truss reaction.

Diagonal bracing, without horizontal cross-struts, was chosen in preference to a rectangular system. For the same weight of metal, it results in a higher degree of rigidity. In the opinion of the authors, it gives a better expression of this rigidity and of the resistance to lateral forces. For the sake of appearance, the widths of the bracing members were increased beyond those required by stress considerations.

Stress calculations

The floor and hangers of the suspension system were designed for the live-load stresses resulting from truck and interurban car concentrations. All other parts were designed for a live load of 4,000 lb. per lin.-ft. per cable, with the additional assumption that the unbalanced live load will exceed 3,000 lb. per ft. The average dead load of the suspended structure, including cables, is 20,000 lb. per lin.-ft. The two extreme conditions, not considering lateral forces, result in a movement of nearly 7 ft. at the top of tower 2.

With live loads distributed as shown in sketches for case I and case II (Fig. 3), the resulting deflections of tower tops under assumptions that would produce maximum variation would be as given in the following table:

TOWER TOP DEFLECTIONS, IN FEET

	CASE I (Temp. + 92° F.)		CASE II (Temp. + 32° F.)	
	Left	Right	Left	Right
Actual deflection		3.71	2.89	
Initial deflection	0.67		0.67	
Construction error		0.17	0.17	

Net deflection: Right, 3.21 ft.; Left, 3.73 ft.

With such distortions, a comparatively flexible tower was required. The l/r ratio, using the value of r at mid-height, is 70 for towers 2 and 6 and 75 for towers 3 and 5. Each column has a cruciform section. In the original

design the central portion was divided by vertical diaphragms, as are the other four parts of the tower leg. This was changed primarily to reduce field riveting and to throw the greater amount of metal to the outer sides of the tower legs. This change, too, made it possible to make use of the open center well for supporting the hammerhead used in erection.

Silicon steel was used in general throughout the columns. A maximum unit stress of 28,000 lb. per sq.in. was permitted for the combination of direct load and bending. In the bracing and minor details carbon steel was used with permissible working stress of 22,000 lb. per sq.in. A 10 per cent increase in stresses was permitted to carry wind loads, and a 40 per cent increase was allowed in carrying earthquake loadings.

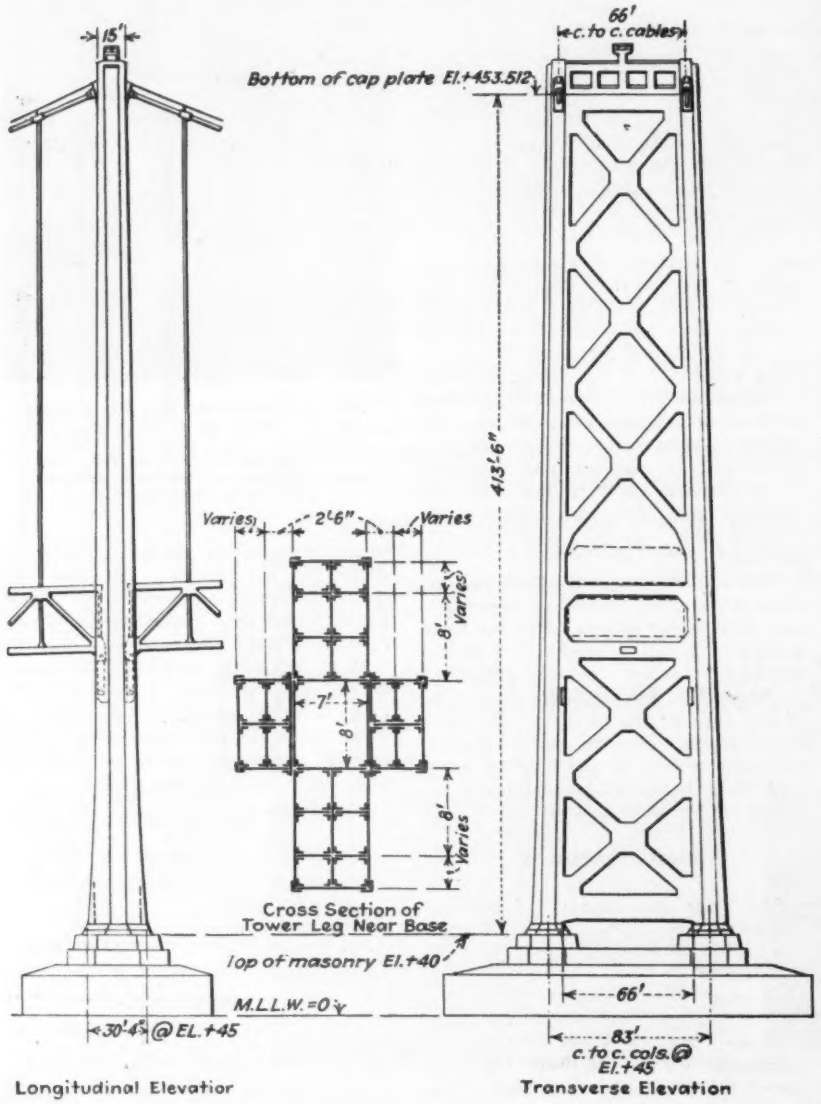
In the tower design the vertical reactions from the cables and stiffening trusses and the horizontal cable movements at the top of the tower were calculated. Sections of the tower were assumed. The horizontal force was then found at the top of the tower, as would be necessary in combination with the other forces to deflect the tower an amount equal to the cable movement. With this force known, the stresses were calculated and the tower sections revised, if necessary. This process was repeated until the desirable balance was obtained.

The placing of the material in the most efficient location was a matter of rather delicate balance. Material near the neutral axis reduced the stresses caused by direct loads but had little effect on those caused by bending. The bending stresses could only be reduced by placing material farther from the neutral axis. This, however, increased the tower stiffness and the unbalanced horizontal force at the top of the tower. An extreme case of this occurred at tower 2 where, on account of the large motion of the tower top, the bending stresses were more severe than at the other towers. In this case reinforcing plates were added at the locations shown by Fig. 3. A more economical distribution of metal might have been obtained, the arrangement shown being used to secure shop duplication since, aside from this reinforcement, towers 2 and 6 are alike.

Ordinarily deflection riverward is greater than shoreward deflection. The vertical loads also are greater in the first case. In order that unit stresses may be approximately equal in each extreme position, in the case of tower 2 the cables will be strung so that the tower has a shoreward deflection of 8 in. A tolerance or allowance of 2 in. in either direction was made to compensate for inaccuracies in the initial deflection.

Stresses in the tower also were calculated for transverse loading from a 30-lb. wind and from earthquake. In a suspension bridge, transverse stresses from seismic forces are comparatively small. The stresses from a seismic shock of 0.1g in both longitudinal and transverse directions also were calculated. In the latter case, the inertia of the suspended structure could result in a considerable force exerted at the truss level.

FIG. 2—FRONT AND SIDE ELEVATIONS and section of a typical tower of the San Francisco Bay suspension bridges. Note open well in center of tower, which concentrates metal in outer part of section.



Secondary stresses

As the tower columns shorten under load the vertical distance between the intersections of the bracing diagonals decreases. This results in putting part of the vertical load in the bracing and in bending both the columns and the bracing members.

As a matter of complete analysis, the longitudinal and bending stresses in the tower members were calculated, taking in the effect of these distortions. (For method, see "Williot Equations for Indeterminate Structures in Combinations with Moment Equations in Terms of Angular Displacements," *Proceedings*, Am.Soc.C.E., January, 1934. The method used in this case is described in discussion by R. A. Tudor, *Proceedings*, Am.Soc.C.E., September, 1934.) These calculations were checked by celluloid model analysis. In no case did the secondary stress in any section of the tower columns exceed 10 per cent of the primary stress.

The absence of horizontal cross-struts allows the columns to bow out under load, so that the longitudinal stresses in the diagonals due to vertical load are comparatively small. In fabrication, the lengths of the diagonals were made such that the column axis will be straight under full load.

In the development of the design, certain details were worked out that differ

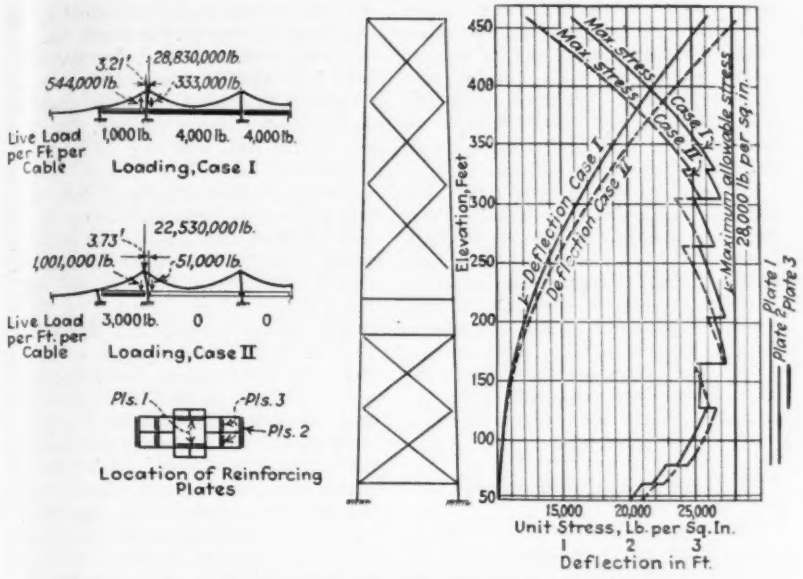


FIG. 3—SKETCHES ILLUSTRATING two assumed live-load distributions and a diagram of stresses and deflections for tower 2.

from previous practice, as noted in the following:

1. The base plates are 4-in. rolled-steel plates. The connection between the tower columns and these plates was made by field welding. With this arrangement the number of connection angles was greatly reduced. The detail could have been still further improved by omitting all connection angles and depending entirely on welding.

2. Vertical diaphragms or cross-webs in the center of the column section were omitted. This obviated the usual entering joints and reduced field riveting to a minimum. The field rivets averaged 110,000 per tower, with 5,000 tons of steel per tower, or 22 per ton. At the base and top of the tower, the center well has diaphragms, to distribute the load over base and cap plates.

3. The batter of the towers and the flare of the column sections were made such that the inside faces of the two columns below the roadway are vertical and 66 ft. apart. This permitted the girders carrying the stiffening-truss reaction to be secured to the inside faces of the columns.

4. The wind reaction is taken by a pin and a rectangular longitudinal sliding detail at the lower roadway strut in each tower.

Personnel

The contract for the towers is held by the Columbia Steel Co., of which Ambrose Diehl is president and E. J. Schneider contract manager. Steel is being fabricated and erected by the American Bridge Co., of which C. S. Garner is general manager of erection and H. C. Hunter is western erecting manager. The personnel of the California Toll Bridge Authority was given in *ENR*, March 22, 1934, p. 377.

Hammerhead Derricks Erect Steel Towers of Bay Bridge

Substituted for the usual creeper travelers, the derricks were better adapted to special conditions obtaining on the 500-ft.-high towers of the San Francisco Bay bridges—Counterbalance used for heaviest loads, which weighed 78 tons

By H. C. Hunter
Western Division Erecting Manager, American Bridge Co.

ON THE San Francisco-Oakland Bay Bridge the method of erecting the steel towers was materially affected by the fact that the tower legs incline toward each other and decrease in cross-section as they rise. In studying the plan of erection it was apparent at the outset that if use were made of the usual creeper travelers, which could be moved up the outer surface of the towers as erection proceeded, numerous problems would be involved and doubtless there would have to be much scheming and many special connections in the tower sides. Search for some better erection plan focused attention on the 7x8-ft. well, which extends uniformly throughout each leg of the towers except for short sections at top and bottom where vertical diaphragms are used. "There ought to be a way to use those holes," it was said, and out of that suggestion grew the plan of supporting the erection rig consisting of a hammerhead derrick entirely within the tower leg.

Not only was the top cross-arm (common in hammerhead derricks of navy yard practice) well adapted to lifting and placing successive parts of the tower, but the central wells provided an almost ideal footing support for the mast (on which the cross-girder rotated) and also greatly simplified raising the rig from point to point as erection progressed. A plumb mast in an inclined well was made possible by beveling one side of the mast near the bottom; the horizontal diaphragms at 12½-ft. intervals in the wells were placed after the derrick had moved past, and a heavy counterweight was used when picking up the heaviest loads. Altogether, features of the erection program were developed so that in the field the work advanced smoothly and at a rate such that two of the towers were erected in only about half the time which their construction probably would have required if creeper travelers had been used.

Features of design

Except for the top and bottom sections, steel for the tower legs was delivered to the job in 50-ft. lengths built up into the prescribed cross-sections. One of these lengths placed on each of the four sides of the 7x8-ft. central well made up a 50-ft. lift of the tower leg.

The main mast of the hammerhead was 108 ft. long. This was sufficient to give a 50-ft. supported length below the top of steel at a time when the hammerhead would be placing the next 50-ft. lift, with sufficient room for the falls between the crosshead and the new addition to the tower leg. Horizontal joints in the two sections parallel to the bridge centerline were 12½ ft. above joints between sections on the opposite axis. These field splices were used for the supports on which the hammerhead mast was carried, thus eliminating the

necessity for special connections to attach the hammerhead rig to the tower.

If the mast of the hammerhead were to stand plumb, it could not be made the full size of the well cross-section because the tower legs (and consequently the well, also) slope toward the centerline of the bridge at the rate of ¼ in. in 12 ft. In order to keep to a minimum the reduction in section made on this account, the inner face of the mast for 30 ft. from the bottom was given a corresponding bevel.

The heaviest piece to be hoisted weighed 78½ tons. This load had to be picked up at a radius of 16 ft. from the center of the well. However, after being lifted at this radius to a point where the load could be moved in toward the tower, a 10-ft. radius of rotation was sufficient for clearance in the process of placing any part. Therefore, 80 tons on a 10-ft. radius was taken as the governing load in design of the hammerhead and, to provide for the lift on a 16-ft. radius, a counterweight was placed under the rear end of the hammerhead, so that the moments on the mast were no greater on the 16-ft. radius with the counterweight than on a 10-ft. radius without it. Loads were never rotated while the counterweight was in use. In fact, it never was raised more than a few inches off its support, the weight being thus carried only while the load was lifted and drawn into the 10-ft. radius, after which the counterweight was again rested on its support before rotation began.

In order to handle floor struts and diagonal bracing between the legs, lesser loads were handled at radii up to 20 ft. The position for picking up loads was on a line midway between the transverse centerline of the tower and a line parallel to the centerline of the bridge. Thus, to set the steel in place on any of the four sides of the tower, the hammerhead was revolved either 45 deg. or 135 deg. from the position in which the load was picked up.

Make-up and dimensions

The hammerhead mast was 5x5 ft. in section, made up of 4x4x½-in. angles at the corners with ⅜-in. steel plates on all four sides. Attached to the bottom of the mast were two girders that spanned the 8-ft. dimension of the well; these rested upon and were bolted to brackets, which in turn were bolted to the face of the well at the field splices. By these girders and brackets the mast not only was supported, but its bottom was held against horizontal movement. With the bottom of the mast supported at the topmost complete field splice, the top of the well was 37½ ft. farther up the mast. Because of this spacing between field splices, the mast was equipped with screw jacks built in on all four sides 37½ ft. above its lower end; the jacks were used to plumb the mast and rigidly wedge it in place at the top of the well.

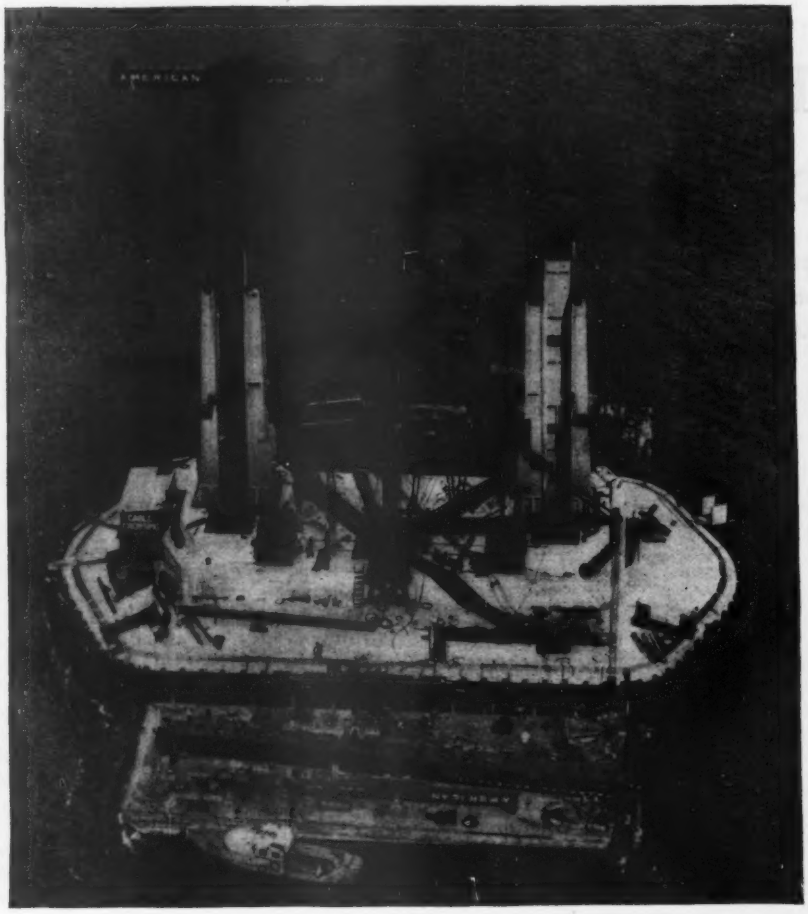
A rotating mast 3¼ ft. square and 24½ ft. long, carrying the cross- or trolley-girders, was mounted inside the upper portion of the main mast and was made up of four 8x8x½-in. angles with ½-in. plates on all four sides. At the bottom the rotating mast was supported on a combination step and ring bearing, and at the top by a ring bearing in the extreme top of the main mast. The step bearing contained a bronze washer and ring on which the steel foot of the rotating mast turned. As the diameter of the ring bearing at the top had to exceed the diameter of the rotating mast and therefore carried only low pressures, steel castings were used for both parts. All bearing parts were sweated with hot paraffine before assembly and were equipped with high-pressure grease fittings.

The cross- or trolley-girders were bolted rigidly to each side of the rotating mast at its top and extended 28½ ft. from the mast in each direction. Their webs were constant throughout their length (7/16 in. thick and 48 in. deep), but as the counterweight was sufficient only for a moment equal to the difference between the 10- and 16-ft. radii, the cross-girders had lighter flange angles at the rear than at the front. The two girders were connected and braced with diaphragms and laterals at the rear, but as it was necessary for the space between them to be clear at the front to pass the lifting cables, connecting bracing there could not be used. The flanges in front, therefore, were stiffened by horizontal wing plates. These plates were 30 in. wide at the top flange and 20 in. wide at the bottom flange. These plates tapered toward the outer end and were strengthened by 3½x3½x⅜-in. stiffening angles.

The front half of the cross-girders had 60-lb. standard railroad rails mounted on the top flanges throughout their length, making a track of 39-in. gage. On this was operated a car supported on four double-flanged wheels and carrying a shaft on which were six 20-in. ball-bearing sheaves constituting the upper block of the main lifting falls. Upon the rear end of the car two smaller sheaves were used for auxiliary lifting lines.

The main lifting falls consisted of ten parts of 1-in. wire rope passing through the sheaves of the trolley at

FIG. 1—HAMMERHEAD DERRICK being assembled in base of tower leg by the stiff-leg derrick that later unloaded steel from barges onto the tower foundation in position for erection.



the top and through a five-sheave ball-bearing block at the bottom. After passing through both these blocks the two ends passed horizontally along the top of the cross-girders, around sheaves over the center of the mast, down through the mast and center well to the foot of the tower and then horizontally through holes in the webs to the hoisting engine. The auxiliary or runner lines, of $\frac{5}{8}$-in. diameter, after passing over the rear sheaves of the trolley, followed the same course to their hoisting drums at the base of the tower. The function of the auxiliary falls was to provide a higher lifting speed (up to 800 ft. per min.) that would save time in handling light loads.

Lines from the main and auxiliary lifting falls running horizontally from the trolley to the center of the mast always tended to pull the trolley toward the center. This was offset by two 1-in. cables attached to the front of the trolley and running around the sheaves at the front ends of the girders, then back to a set of five-part falls attached to the rear ends of the cross-girders. From these latter falls, lead lines passed down through the mast the same as the lifting lines. Thus, taking in on this set of falls moved the trolley away from the center, and slacking them off permitted the lead lines to pull it back.

The inner mast and the cross-girders were revolved by two motors driven by compressed air and connected through suitable reduction gearing to a driving gear or rack which was an integral part of the lower step and ring bearing of the revolving mast. These motors were reversible and, while they developed 5 hp. forward and only 3 hp. in reverse, they were connected so that for either direction of revolution one motor ran forward and the other in reverse. Thus, the same torque was exerted in either direction. The controls were so interconnected that one lever operated both motors. Controls were mounted inside the main mast just below the revolving portion. An extra large hole or doorway in the side plates of the main mast at this point gave the operator an unobstructed view of operations below. At this point, also, were operated the signals to the hoisting units, so that the man at the revolving controls served as signalman for all other operations.

All steel raised by the hammerhead derrick was picked up at the same point. Thus it was possible to place the counterweight, a 17½-ton concrete block, at a point such that it always was immediately beneath the rear end of the cross-girder. From the counterweight a 1¼-in. wire rope passed over a sheave at the rear end of the cross-girder and thence to a set of falls anchored to the pier. With these falls the counterweight could be raised a few inches from its base, thus putting a 35-ton load on the rear of the hammerhead. With the counterweight in service the lifting falls could carry an 80-ton load on a 16-ft. radius while the bending moment on the mast was kept within the prescribed limit of 800 foot-tons.

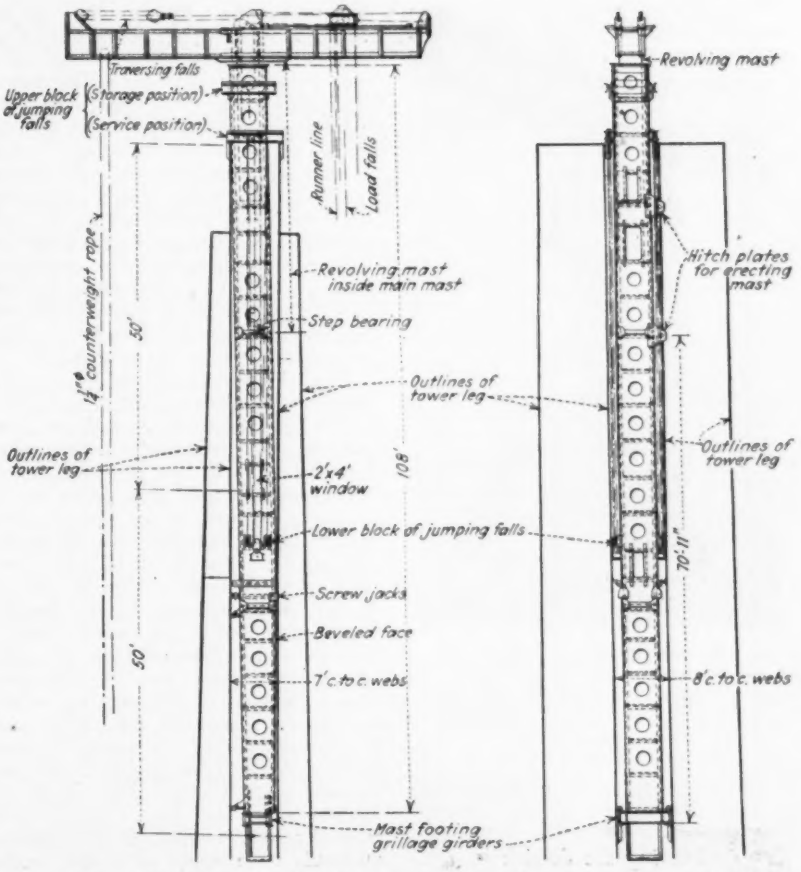


FIG. 2—HAMMERHEAD DETAILS showing main and revolving mast, cross-head and trolley, jumping falls, etc.

Erection procedure

At the start of operations a large stiff-leg derrick was set up on the pier between the bases of the tower legs. Steel was delivered alongside the pier on flat cars by large ferry barges. The 85-ft. boom of the derrick could lift the steel from the cars and was long enough to erect the first section of the tower. With base plates and tower-leg sections up to a height of 75 ft. on three sides of the central well, a setting was ready for the hammerhead derrick. The hammerhead was delivered completely assembled, except for wire-rope rigging, on a barge alongside the pier. From the barge it was picked up bodily by the stiff-leg and set on the base plates in the recess formed by the three sides of the well. The remaining side of the well up to the 75-ft. elevation was then placed. Finally, the hammerhead was raised by the stiff-leg so that the bottom grillage girders could be bolted at the first field splices. Here the rigging of the hammerhead was completed.

Throughout erection the hammerhead was served by the stiff-leg which lifted steel from barges alongside, raised the sections to a vertical position and set them up within convenient reach. Hitch plates attached to the top of the sections were so arranged that they could be used both by the derrick for upending the sections and by the hammerhead for hoisting them to position.

The uniform 7x8-ft. dimensions of the center well gave all sections of the tower legs one side that was either 7 or 8 ft. wide. Hammerhead and derrick alike, therefore, were equipped with balance beams that could readily be adapted to one or the other of these dimensions.

Originally it had been the intention to use the counterweight only for sections weighing more than 50 tons. It was found, however, that the counterweight could be picked up and landed so readily that it was decided to use it in every case, thus avoiding the risk of forgetting it for heavy loads. In order to minimize interference, the sequence of erection was to place a section 135 deg. to the right, then a section was swung through the same angle to the left, and finally a section 45 deg. each way. Each hammerhead handled the diagonal bracing attached to the leg in which it operated; both hammerheads worked in conjunction when placing horizontal struts and floor girders. The largest though not the heaviest section handled was the horizontal bottom strut approximately the size of a pullman car.

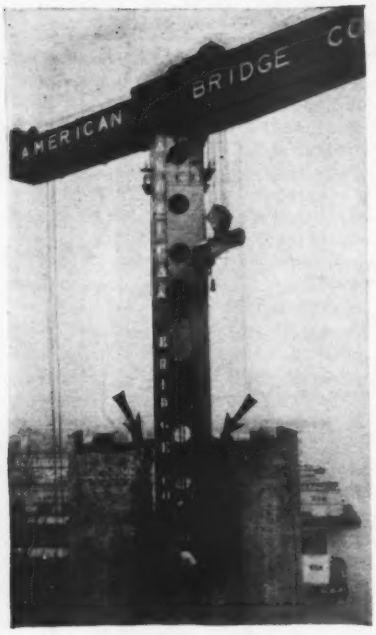


FIG. 3—HAMMERHEAD in upper tower leg. Balance beam is suspended from main falls; rotating mast is seen through opening in upper main mast. Arrows indicate transverse beam for upper block of jumping falls.

It was 8 ft. wide by 14 ft. deep and 65 ft. long; its weight was about 70 tons.

Moving the derrick

The hammerheads were raised from point to point as erection proceeded by jumping falls placed on opposite sides of the mast. These consisted of eight parts of ¾-in. wire rope. The lower sheaves were built into the lower end of the main mast, and the upper sheaves were part of a small cross-girder assembly placed on either side of the mast on top of the last tower section to be erected in each 50-ft. lift. To insure both of these sets of falls getting equal amounts of the load, only one piece of wire rope was used in reeving up the combination. That is, this rope ran continuously from the hoist, through one set of falls, then through the other falls and back to the hoist.

To facilitate movement of the mast in the process of raising, rollers and runners were placed where the mast would bear against sides of the well. These were so located that they would move between the vertical lines of rivets in the tower sections. Large holes in the side plates of the mast gave workmen access to the interior for rigging, operating the jacks, etc. Ladders and platforms throughout its length were provided.

After a 50-ft. tower height had been added (an operation which included placing four 50-ft. sections on each leg) the attendant bracing was put in and sufficient bolts were placed to hold the assembly rigidly in position. The small girders carrying the top block of the lifting falls then were placed across the top of the steel, which at that time was about 8 ft. below the cross-girders. The jacks wedging the upper end of the mast in the well were released, and the bolts holding the bottom grillage girders to their supporting brackets were slacked off. The entire hammerhead assembly then was raised about 1 in. by the lifting falls to take the strain from the supporting brackets bolted to the tower splices. After this the bolts holding these brackets could be removed, and the brackets were allowed to hang from the grillage girders by the loose bolts.

Further operation of the lifting falls raised the hammerhead 50 ft. to a point where the supporting grillage and brackets could be bolted into the splice bolt holes at a new working level. While the brackets were being bolted in place, the mast landed and plumbed up and the jacks tightened at its upper bearing, the comparatively light horizontal diaphragms designed for the portion of the center well just vacated by the mast were raised by the auxiliary hoisting lines and lowered into the well alongside the main mast to their designed location. Placing of these diaphragms completed the cycle, and the hammerheads were ready to start another tier of tower section. On the average slightly less than 2 hours elapsed from the time erection was stopped at one elevation until the hammerheads were ready to resume work in the next position 50 ft. above.

Upon completion of the tower proper a 25-ton guy derrick was erected on the top strut. This derrick was used to dismantle the hammerhead and to place the final diaphragms, cap-plates, etc., which could not be put on until the hammerheads were out of the way. As the saddle castings capping the top of the tower legs weighed 46 tons, which was more than the guy derrick could lift, they were hoisted while the hammerheads were still in service and temporarily were placed on the top struts adjacent to final position. Later, when their settings were ready, the saddle castings were jacked sideways into final position.

Contingencies that did not originate with erection procedure materially delayed progress on the first and the fourth towers. On the second and third towers the erection periods each were about three five-day weeks, using a twelve-hour working day. In this time erection was completed from the 75-ft. elevation, where the hammerheads were put into operation, up to the dismantling of the hammerheads. Each of the towers contains more than 5,000 tons of steel.

Four 175-hp. gasoline hoisting engines were used on the job, two for each hammerhead. Each of these engines had two main hoisting drums with a lead line pull of 21,000 lb. and two auxiliary drums capable of a 10,000-lb. pull. The main drums held 4,400 ft. of 1-in. wire. One main drum on each of the pair of hoists assigned to a hammerhead handled the main load line. The other main drum of one hoist handled the traversing line, while the corresponding drum of the other hoist was equipped with a dividing flange in the center so as to handle both ends of the jumping falls used for raising the hammerhead. Auxiliary drums were used for handling counterweight, auxiliary hoisting lines, etc.

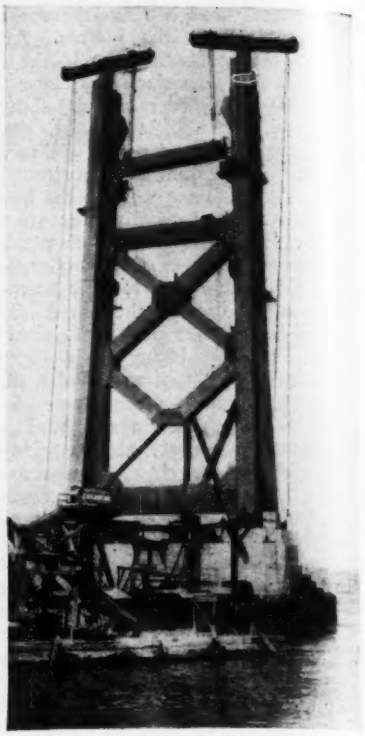


FIG. 4—HAMMERHEADS making joint lift, to place upper-floor strut in tower No. 2.

Only one tower was in process of erection at any one time. To facilitate moving the hoists from tower to tower, all four of them were permanently mounted on a large steel barge, which was moored successively at the several piers on the side opposite that on which the steel was delivered by ferry. In order that tidal variations would have only a slight effect on the lines in the hammerhead derricks, the lead lines were arranged to pass horizontally for quite a distance from the foot of the tower before being led to the hoist on the barge.

Contract for the entire superstructure of the San Francisco-Oakland Bay Bridge is held by the Columbia Steel Co.; fabrication and erection are in the hands of the American Bridge Co.; both companies are subsidiaries of the United States Steel Corp. Erection is under the direction of C. S. Garner, general manager of erection, and the writer. E. E. McKeen is resident engineer, and W. J. Ward, superintendent, is in charge of field operations.

Tests Indicate Design Methods for Earthquake-Proof Timber Floors

A summary of recent full-size and quarter-scale model tests of wood floors considered as diaphragms in masonry wall buildings

By Norman B. Green, A. C. Horner and Theodore C. Combs
Consulting Engineers, San Francisco, Calif

THE PROBLEM of earthquake or wind-force design of the framework structure of a masonry bearing-wall building includes the following elements:

1. Design of the side walls in such a manner that stresses set up therein by the earthquake force normal to the walls will not produce a deflection, *d*, greater than that which can be allowed within safe working stresses on the masonry.

2. Design of a diaphragm which will transmit stresses from the side walls to the end walls without exceeding a deflection *d*.

3. Design of a proper anchorage of the diaphragm to the side walls, so as to resist the tensile and compressive forces involved.

4. Design of the connection of the diaphragm to the end walls, so as to resist the reaction or shear forces, *V*.

5. Design of the end walls in such a manner as adequately to resist shear forces *V*.

It is with item 5 above that this article chiefly is concerned. Information on the various elements of design listed has been secured in a series of tests made by the authors and others, which were initiated in December, 1933, and sponsored by groups of retail lumber dealers in Long Beach and Los Angeles, Calif.; by the National Lumber Manufacturers Association; and by the Clay Products Institute of California. Also certain of the tests referred to herein—namely, those on the 37th St. School—were originated by the architects' and engineers' advisory committees of the division of architecture, state of California, and were carried out under the direction of Frederick J. Converse, of Labarre & Converse, foundation and testing engineers, Los Angeles.

As originally planned and outlined in *Engineering News-Record*, Feb. 1, 1934, p. 142, in an article describing the first series of tests in Long Beach, the test program was composed of the following three parts:

1. Lateral tests of full-size floor panels with varying arrangements of flooring and subflooring.

2. Lateral tests of one-quarter-scale-model floor panels, first duplicating the full-size tests, and second introducing other variables such as ratio of span to length, size and arrangement of joists with respect to direction of applied load, size and number of nails, size and arrangement of flooring and subflooring, and effect of lapping joists.

3. Tension and shear tests of bolt anchors in brick and tile walls varying in type, size and arrangement of anchors, as well as in thickness and mortar mix of both the brick and tile walls.

As stated, the tests for part one are reported in *Engineering News-Record*, Feb. 1, 1934, p. 142. The third series of tests, performed in the laboratories of Stanford University, was reported in *Engineering News-Record*, July 12, 1934, p. 35. The second and final part of the test program is reported herein and includes tests made in San Francisco on quarter-scale models, and tests made on full-size floors in the 37th St. School in Los Angeles.

Details of design

Study of the results of the various tests on timber floors acting as diaphragms in masonry-wall buildings have led to certain conclusions with respect to details of design. They affect the side walls, the diaphragms themselves and the wall anchorages.

Side Walls—The side-wall design may be treated empirically, by simply assuming a maximum permissible lateral deflection in each story. This may be arbitrarily set at ¼ to ½ in., depending upon the height of the story. Under such conditions the masonry will crack but, unless the cracks are of considerable magnitude and extent, friction will hold the masonry in position.

The wall may also be designed rationally, so that it will not crack, considering it as a vertical beam carrying a lateral load due to its inertia, with restraint at the ground and supported on one or more yielding reactions at the roof and floors. For those unusual cases in which the length, *L*, of the side wall is less than about three times its height, *H*, it becomes necessary to consider also the stresses due to longitudinal beam action of the wall acting as a restrained beam of span, *L*, between the two end walls. This problem was treated at some length in a previous article (*ENR*, Feb. 1, 1934, p. 142).

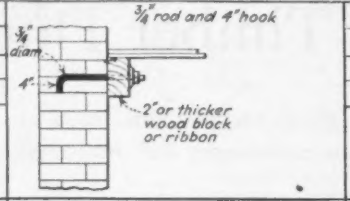
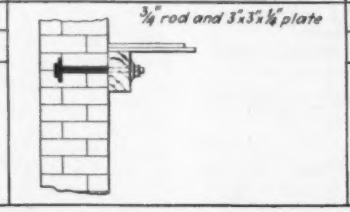
The Diaphragm—The design of the diaphragm entails the determination of a type of construction which, under given conditions of span, *L*, breadth, *b*, and total uniform force, *F*, will limit maximum deflection, *d*, to an amount not greater than that allowable for the side wall. Accordingly, the selection of side wall and diaphragm are interdependent. A general formula has been derived further on in this paper, which gives the lateral deflection of a "Standard" type of wood floor as a function of the span, width and loading. For convenience in design, Table 4 has been computed from this formula and Fig. 4 has been plotted, to give deflections produced by a unit uniform load on floor panels of any dimensions.

Wall Anchorage—An efficient wall-to-floor connection consists of bolting a timber "ribbon" to the walls parallel to the joists (end walls) and bolting solid blocking (cut-in between joists) to the walls perpendicular to the joists (side walls). There would probably be one bolt used for each cut-in block; the bolt spacing for the ribbon may be the maximum allowable for the forces to be resisted. Diagonal flooring would be nailed to both the ribbon and blocking with sufficient nails to develop the necessary resistance to tension, compression and horizontal forces parallel to the wall. This construction has the added advantage of providing a practically continuous support for the wall, so that lateral bending of the wall between anchors is largely eliminated, and with it also the necessity for a bond beam.

The side-wall anchorage is called upon to resist stresses of direct tension or compression set up by forces normal to the wall, arising from the action of the distributed lateral load, *F*, (or more strictly, that portion of the total load, *F*, produced by the inertia of the wall itself). In the third column of Table 1 are given recommended tensile working loads for the anchorbolts that connect the blocking to the side wall.

The end-wall anchorage is called upon to resist the shearing force, *V*, acting parallel to the wall. The fourth column of Table 1 gives recommended working loads in shear for the anchorbolts that connect the ribbon to the end wall. It should be noted that when the direction of the ground motion is parallel to the length, *L*, of the building, the function of the wall anchorages is reversed, so that the end-wall connection must resist tension and compression while the side-wall connection resists a shear force.

TABLE 1 RECOMMENDED SAFE LOADS FOR ANCHORBOLT ASSEMBLIES

TYPE AND THICKNESS OF WALL	SIZE AND TYPE OF ANCHOR	SAFE WORKING LOAD, LB.	
		TENSION	SHEAR
Brick - 8"		1,500	1,180
Brick - 12"		2,300	
Tile - 8"		1,300	
Tile - 12"		2,200	
Brick - 8"		3,000	1,180
Brick - 12"		4,600	

NOTES:
This table is based on data obtained in the Stanford tests (*ENR*, July 12, 1934, p. 35). A factor of safety of 4 has been used for safe-load values in tension. Safe-load values in shear are limited by and based on the bearing value of Douglas fir under bolts. (*Technical Bulletin* 332 of the U. S. Department of Agriculture.) Average workmanship is assumed.
The safe loads shown are based on a 1:1:6 cement-lime mortar laid up in the customary manner.
Washer sizes for bolts in tension will vary according to allowable stress in bearing perpendicular to grain.

As a part of this test program, it seemed desirable to investigate the effect of varying lengths, widths and areas of diaphragms on their rigidity (as measured by deflection under lateral load). Previous tests seemed to indicate that a wood-floor panel loaded laterally deflects in a manner analagous to a solid beam, the total deflection being the sum of the deflections due to shear and bending.

In order to check the correctness of this observation and to establish a general relation between load and deflection for varying ratios of length to breadth, L/b, an investigation was carried out in San Francisco by the authors on a quarter-scale model of one of the floor panels tested at Long Beach.

Quarter-scale model tests

The Long Beach prototype was 24 ft. wide and 40 ft. long, with 2x12-in. joints laid crosswise of the panel, 16 in. apart. There were two layers of boards, the lower layer (subfloor) of 1x6-in. x 12-ft. boards laid at 45 deg. to the joists, with ¼-in. cracks between boards, and nailed with two 8d common nails at each butt; the upper layer of 1x4 in. x 14-ft. square-edged boards was laid at right angles to the joists without cracks between boards and nailed with two 8d finishing nails at each joist (not blind nailed). The butts of adjacent boards were staggered two joist spaces. This is the so-called "Standard" floor.

The model tested was an exact reproduction, on a quarter scale, of the Long Beach panel in each linear dimension of each member, including nails.

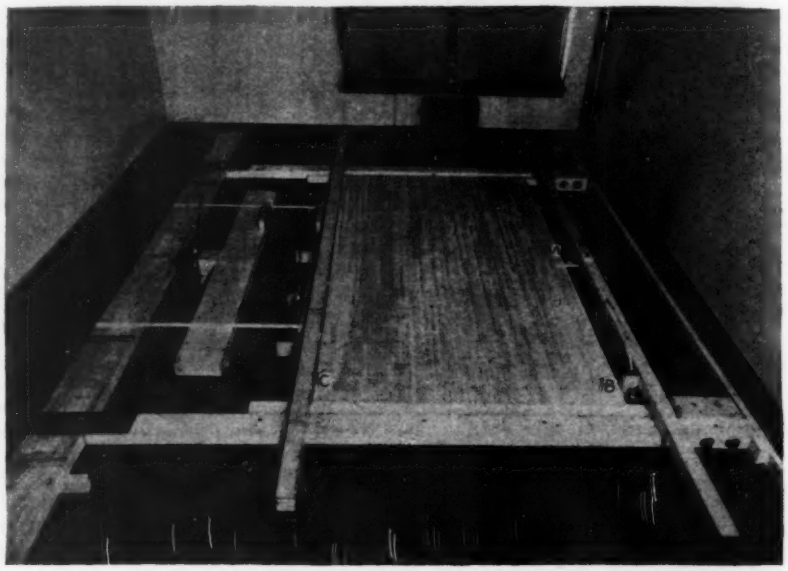
The test apparatus is shown in Fig. 1. The lateral load was applied by means of screw jacks and measured with a calibrated spring. The ends of the panel rested against metal-toggle-type bearing blocks that permitted an unrestrained rotation of the ends of the panel, as well as a change in the distance between supports. The deflection at the center of the panel was measured by means of a dial gage (A), reading to thousandths of an inch. It was found necessary, in some of the tests, to place weights on the panel at certain points to prevent upward buckling; but inasmuch as small weights were found to be sufficient, their frictional effect on the recorded lateral loads was neglected.

TABLE 2—LIST OF QUARTER-SCALE MODEL TESTS

Test No.	Panel Dimensions Length, L	Breadth, b	Ratio L/b	Load Applied At
101	10'-0"	6'-0"	1.67	5th points
101-A	10'-0"	6'-0"	1.67	Center point
102	10'-0"	5'-0"	2.0	5th points
103	10'-0"	4'-0"	2.5	5th points
104	10'-0"	3'-4"	3.0	5th points
104-A	10'-0"	3'-4"	3.0	Center point
105	8'-4"	3'-4"	2.5	5th points
106	6'-8"	3'-4"	2.0	5th points
107	5'-0"	3'-4"	1.5	5th points
108	3'-4"	3'-4"	1.0	5th points

All the tests were carried out on a single panel of an original size of 10x6 ft., this being successively cut down during the progress of the tests to the sizes shown in Table 2. In each of the tests the load-deflection curve was plotted during the progress of the loading, and the maximum load was kept below a value that would produce a break in this curve, so as to obviate the possibility of a permanent injury to the panel before the next test. As is shown later, the previous loading of the panel seemed to have no effect on its manner of action in subsequent tests up to and including test 105. In making test 106 the panel was probably overloaded, and the data from tests 106,107 and 108 are subject to doubt. This irregularity may have been due to overloading during test 106, or it may be due to a combination of circumstances. A slight buckling of these small panels has a large relative effect on the dial readings. The small panels were very much stiffer than large panels having the same proportions, which is very likely due to the relatively fewer number of board splices. In order to get any uniformity of action for a built-up construction of this type, it is evidently necessary to have a large enough floor area, so that the board splice pattern will give a definite average elastic action for the

FIG. 1—TESTING A QUARTER-SIZE MODEL of a floor panel, to determine relation between length-width ratio and rigidity under lateral load.



floor as a whole. Because of the above uncertainties in tests 106, 107 and 108 the results of these tests are not included.

Analysis of model-test results

If it is assumed that a built-up floor of this type, while not in any sense a homogeneous structure, has nevertheless definite elastic properties when considered as a whole, it is practicable to treat it as a beam and to employ the usual methods of beam analysis. On this basis, from the elastic theory of beams, its lateral deflection in inches under a fifth-point loading may be expressed as:

$$d = \frac{3WL}{20bE_s} + \frac{WL^3}{5.3b^3E_m} \quad \ldots\ldots(1)$$

in which the first term represents the deflection customarily considered as that due to shear, while the second term is that due to bending. In this equation, W is the total load on the panel in pounds, L the span, and b the width in feet; while E_s and E_m are the shear-modulus and direct-stress modulus respectively, the unit being a foot width of floor. The assumption is made that the shear stress is uniformly distributed over a transverse section of the floor, as it is not believed that the beam analogy is so close as to warrent the refinement of assuming a parabolic distribution.

The corresponding equation for a center load is—

$$d = \frac{WL}{4bE_s} + \frac{WL^3}{4b^3E_m} \quad \ldots\ldots(2)$$

while a uniformly distributed total load W gives the deflection

$$d = \frac{WL}{8bE_s} + \frac{5WL^3}{32b^3E_m} \quad \ldots\ldots(3)$$

Values of E_s and E_m can be determined from any two tests, on different-sized floor panels of the same type of floor assembly, by writing a deflection equation for each test, in which are substituted the dimensions of the panel, along with a given load and the corresponding measured deflection. The two unknown values E_s and E_m can then be determined by solving these equations simultaneously. This has been done in the case of the model-test panels, by taking different tests in pairs; average values of E_s and E_m of 24,900 and 113,700, respectively, have thus been determined ($E_s/E_m = 0.219$).

Using these average values of the moduli in the appropriate equation, (1) or (2), a theoretical load-deflection line has been computed for each test and plotted as a broken line on Fig. 2, in order to give a comparison with the experimental line. It will be seen that, for five-point loading, the measured load-deflection curve is in close agreement with the theoretical, computed

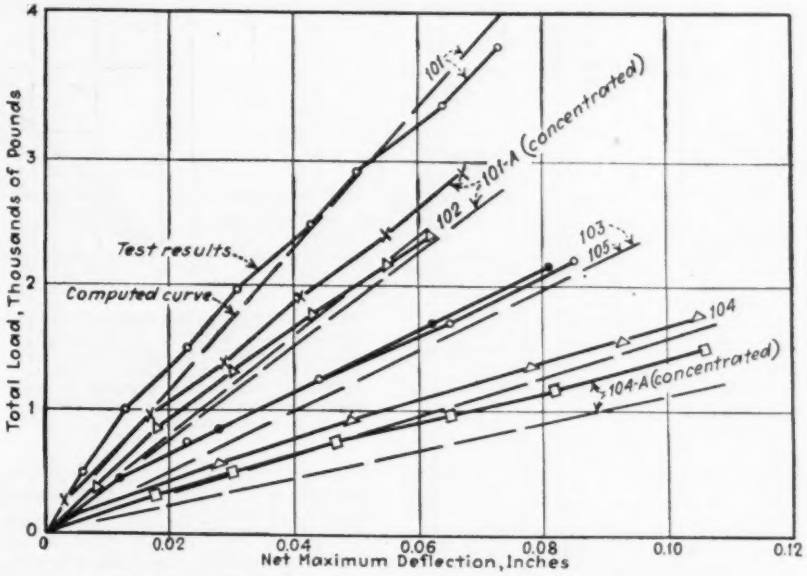


FIG. 2—MEASURED AND COMPUTED load-deflection curves for quarter-scale floor model. Numbers on curves refer to tests listed in Table 2.

load-deflection line. The agreement is not so close in the case of center-point loading, due in part to the method of loading (through the three center joists), which does not give actual center-point loading. In every case the computed deflection is greater than the measured deflection. This discrepancy is due to two things: (1) an upward curve of the actual load-deflection lines near the origin above which, however, the deflection becomes nearly proportional to the load and the resulting straight load-deflection line is in each case quite closely parallel to the theoretical line; (2) the relative effect of end fixity for various ratios of (L/b), which has been ignored in deriving the formula. The effectiveness of the toggle joints used in the tests and intended to prevent end fixity is questionable. The agreement is close enough, however, taking into account the widely varying dimensions of panel and types of loading, to justify the application of the foregoing method of analysis to full-sized floors.

Full-sized floor tests

A list of full-sized scale tests completed at Los Angeles and Long Beach is shown on Table 3. The tests referred to here were carried out at the 37th Street School in Los Angeles, a building that had been condemned (for structural reasons not affecting the components to be tested) and that was made available for test purposes by the Los Angeles Board of Education. Structural elements were typical of those found in any ten-year-old building. (A complete report of these tests is available on request to the authors.)

For the purpose of these tests a classroom floor was selected, having a length of 35 ft. between brick interior partition walls and a width in the direction of the joist span of 23 ft. between the

TABLE 3—LIST OF FULL-SCALE FLOOR TESTS

Test No.	Structural Element Tested	Application of Lateral Load—Direction with Respect to Structural Elements	Points of Application	Primary Purpose	Size, Ft.
LONG BEACH					
LB2	"Standard" wood floor without ceiling	Parallel to floor joists	Ends of joists at 5th points of span	To determine rigidity of this type of floor	24x40
37th STREET SCHOOL, LOS ANGELES					
W1A	Existing wood floor with existing metal lath and plaster ceiling	Parallel to floor joists	Ends of joists at 5th points of span	To determine rigidity of existing floor and ceiling	18x35
W1B	Same as W1A but with plaster ceiling removed	do.	do.	To obtain stiffening value of plaster ceiling	18x35
W2	"Standard" wood floor without ceiling	do	Ends of 3 adjacent joists, at center of span	To determine effect of reversal of loads under both static and dynamic loadings	18x35
W5A	"Standard" wood floor, renailed	do.	Ends of joists at 5th points of span	For comparison of center point loading and 5th-point loading	18x35
W5B	do.	do.	Ends of 3 adjacent joists, at center of span	do.	18x35
W6	W5B repeated but with a width 4 ft. less	do.	do.	To determine effect of variation of width of floor on rigidity	14x35

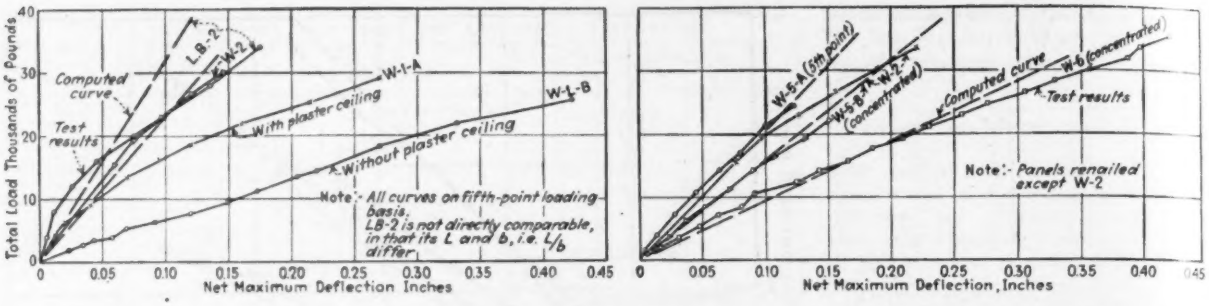


FIG. 3—DEFLECTION CURVES for a full-scale floor panel in a Los Angeles school. Curves W1A and W1B are for original floor. Other curves are for a "Standard" floor that was placed on the existing joists. See Table 3 for list of tests.

exterior brick wall and a brick partition wall along the corridor. The effective width of this test floor was reduced to 18 ft., since a strip 4 ft. wide was cut away along the corridor to provide space for the loading jacks, while another strip 1 ft. wide was removed along the exterior wall, to eliminate restraint at this edge of the floor. The two end (brick partition) walls served as reactions for the panel, the end joist at each wall having been bolted to the brickwork with ¾-in. bolts spaced 16 in. on centers. Deflection measurements were taken at the center of the span and at the quarter points by means of dial gages set between the exterior wall and the unloaded edge of the panel; an additional gage was placed at each end, to record the movement of the two reactions.

The existing floor was first loaded in its original condition, including a metal lath and plaster ceiling, as test W1A. This floor consisted of 2x14-in. joists spaced 16 in. on centers, with a 1x6-in. subfloor laid at 60 deg. with the joists and a 2¼x¾-in. tongue-and-groove maple-finished floor laid at right angles to the joists.

After this test a "Standard" floor was placed on the existing joists, and test W2 was made. The panel was then renailed for tests W5 and W6.

Full-sized test results

On Fig. 3 (left) is plotted the net center deflection of the panel against the total lateral applied load for tests W1A, W1B and W2 and also for the Long Beach panel No. 2.

A comparison of the load-deflection lines for tests W1A and W1B, shows that the plaster ceiling greatly stiffened the panel, and it must, therefore, have carried a large portion of the total applied load. Even under these severe conditions the only cracks that occurred in the ceiling were at the center and quarter points of the span along the line of separation of the metal lath, where the sheets were butted together and tacked to the joists.

With a properly built and very much stiffer floor construction as represented by test W2, the relative stiffening effect of the plaster ceiling would probably be much less.

A comparison of the load-deflection lines for tests W1-B and W2 shows clearly the vital importance of proper construction if a floor is to be used for resisting lateral forces. The rough and finished floor boards must be of proper length, they must be properly nailed, and the butts must be staggered if the floor is to be efficient in lateral bending.

Test W2 may be considered as three separate tests, although the results are plotted as one curve, after being converted to a fifth-point loading basis, for purposes of comparison. There was no progressive increase in the deflection of the panel under the reversd loading, and an examination of Fig. 4 will show that there is no break in the load-deflection line (at 15,800 lb.) when this loading was resumed after the reversed-loading test. This "negative" result is of great practical importance, since it indicates that a floor is not likely to suffer permanent injury or progressive deflection under the action of the reversed loads produced by earthquakes.

On Fig. 3 (right) are plotted the net center deflections of the panels against the total lateral load for the three tests, W5A, W5B and W6.

The deflection equations 1, 2 and 3 may be applied to full-sized floors, if proper values for the shear modulus and bending modulus are first determined. This has been done for the so-called "Standard" floor (Long Beach panel No. 2 and 37th Street School tests panel W2), by substituting the data of test W2 into Eq. (2), which can then be solved for E_s, if E_m is first eliminated by writing it in terms of E_s (since the ratio of these constants should be the same for the model and prototype). On this basis E_s turns out to be 139,000, and E_m is 635,000 ($E_s/E_m = 0.219$), so that the corresponding deflection formula for a uniformly distributed lateral load becomes—

$$d = \frac{WL}{b \times 1{,}100{,}000} + \frac{WL^3}{b^3 \times 4{,}000{,}000} \quad (4)$$

which may also be written

$$d = \frac{wL^2}{b \times 1{,}100{,}000} + \frac{wL^4}{b^3 \times 4{,}000{,}000} \quad (5)$$

in which w is the lateral load per foot length of floor.

It must be emphasized that, so far as we now know, this formula only applies

TABLE 4—NET DEFLECTION OF "STANDARD" WOOD FLOORS UNDER A UNIFORMLY DISTRIBUTED LOAD OF 100 LB. PER FT.

Span (L) Ft.	Width of Floor (b) in Ft.													
	10	15	20	25	30	35	40	45	50	60	70	80	90	100
20	0076	.0036	.0023											
25	0154	.0067	.0041	.0030										
30	0284	.0114	.0066	.0046	.0035									
35	0486	.0185	.0102	.0069	.0051	.0040								
40	0786	.0287	.0153	.0099	.0072	.0056	.0046							
45		.0434	.0226	.0144	.0103	.0080	.0065	.0055						
50		.0614	.0209	.0191	.0134	.0101	.0081	.0068	.0058					
60		.1178	.0568	.0338	.0229	.0169	.0132	.0108	.0091	.0069				
70			.0973	.0563	.0371	.0277	.0207	.0165	.0137	.0102	.0081			
80			.1571	.0889	.0574	.0405	.0305	.0242	.0198	.0144	.0113	.0093		
90				.1344	.0853	.0593	.0440	.0344	.0278	.0199	.0153	.0123	.0104	
100				.1963	.1229	.0843	.0618	.0476	.0382	.0267	.0203	.0161	.0135	.0116
120					.2356	.1583	.1137	.0859	.0676	.0458	.0338	.0264	.0217	.0183
140						.2749	.1946	.1448	.1124	.0742	.0534	.0404	.0330	.0274
160							.3142	.2316	.1777	.1147	.0811	.0600	.0484	.0396
180								.3535	.2689	.1708	.1186	.0864	.0688	.0556
200									.3927	.2457	.1688	.1209	.0953	.0764

to a floor constructed in the same manner as the test floors mentioned above. Nothing is at present known, quantitatively, about the influence upon the stiffness produced by variations in joist spacing, board length, nail size and number or the shift of the butts. It may be said with fair certainty, however, that reasonable variations in the size of the joists, when these run parallel to the applied load, will have little or no effect on the deflections, since the bending and shear stress is resisted by the sheathing acting as a diaphragm, and the joists simply serve as stiffness and to transmit stress through the nails from one board to another. In any case the effect of these variables is not of great importance to the designer, since a floor can be readily built to a given specification.

In Fig. 3 (right) the broken lines represent the theoretical load-deflection lines computed from Eq. (1) and (2), employing the values for E_s and E_m, computed from test W2. While the theoretical line does not fit the Long Beach test data very closely, it averages this curve fairly well up to a load of about 20,000 lb. and gives deflection values on the safe side up to 17,000 lb.

The constants E_s and E_m for full-sized floors have been determined by making use of the ratio E_s/E_m for the model floors, in conjunction with the data on test W2. It will be seen from Fig. 3 (right) that the theoretical load-deflection lines (computed from Eq. 1 and 2) agree very well with the test data of W5A, W5B and W6.

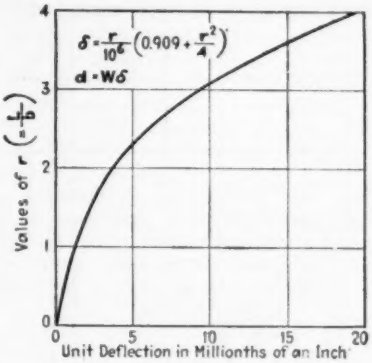


FIG. 4—CURVE for unit deflection of "Standard" floors. To obtain total maximum deflection, multiply unit deflection by total uniform load.

Table 4 gives net deflections, in inches, of "Standard" floor panels subjected to a uniformly distributed lateral load of 100 lb. per foot of span. From the curve on Fig. 5 may be obtained the deflection for any size of floor panel when subjected to a uniformly distributed load of 1 lb.; actual net deflections may be easily determined by multiplying the unit deflection by the total load and are equivalent to the deflections given in Table 4.

Conclusions

1. A wood-floor structure having diagonal and longitudinal flooring, properly laid and nailed to joists running parallel to the applied load, acts to resist lateral bending forces in a manner similar to that of a simple beam, in so far as deflection is concerned. Its deflection can be closely approximated by the use of empirical elastic constants in the usual flexure and shear formulas applicable to homogeneous beams.

2. There is no progressive deflection or apparent injury to the floor panel under the action of repetitive reversed lateral loads, at least up to four complete cycles of stress.

3. A metal lath and plaster ceiling has a very large stiffening effect upon a poorly constructed or inherently flexible wood floor.

4. Proper construction of a wood floor is of the utmost importance in increasing its stiffness, under the action of lateral loads, where the lateral stability of the structure depends on distributing lateral (earthquake or wind) loads from a wall at right angles to the load to walls parallel to the load.

Little Effect of Soil Classes on Optimum Moisture

Four natural and four mixed soils compared as to optimum moisture for equal pressure—Hydrometer analyses of soils compared

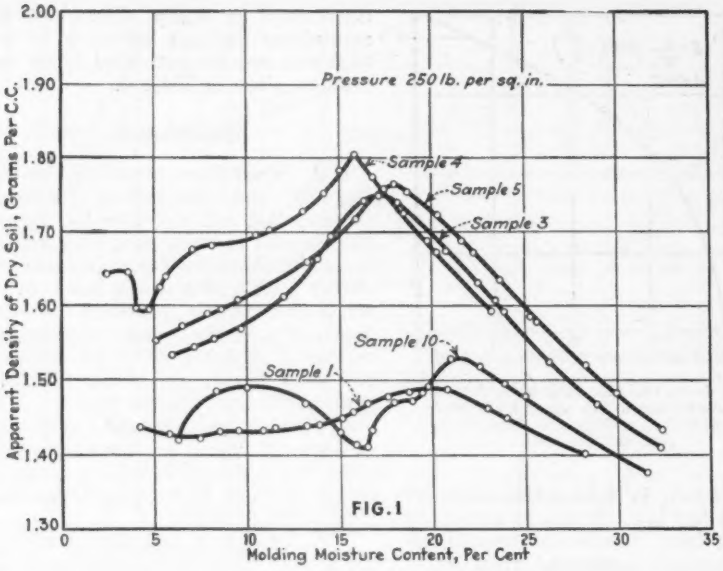
By B. G. Zimmerman
Department of Chemical Engineering, University of Texas, Austin, Tex.

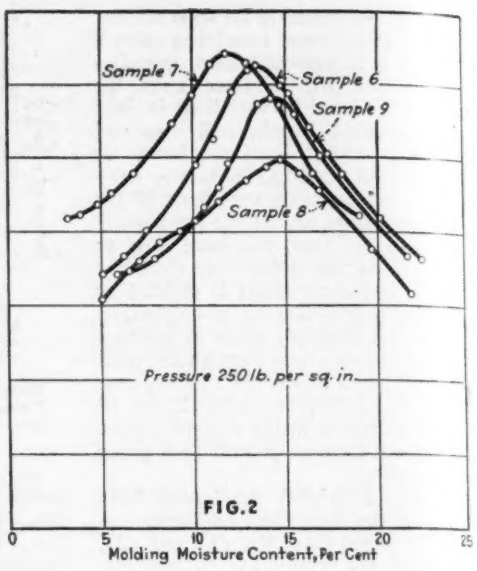
BEFORE the effect of different classes of soils upon the optimum moisture content for soil compaction can be determined, a thorough knowledge of the different classes of soils as applied to foundation work is necessary. Hogentogler, Wintermeyer and Willis (*Public Roads*, 12, Nos. 4, 5, 7 and 8, 1931) of the U. S. Bureau of Public Roads published a complete report of the different soil tests and their use in classification of soils for highway construction. Their classification of soils is used in determining the soil types employed in this article.

Fig. 1 represents the optimum moisture curves obtained from the different classes of soils. Two of the samples (Nos. 4 and 10) were presented in *Engineering News-Record*, June 13, 1935, p. 838, and are classed as A-4 and A-5 types respectively. Since the optimum moisture curves of these two samples, for a pressure of 250 lb. per sq.in., have been determined in the preceding article, the curves are reproduced in Fig. 1. To keep all curves on a comparable basis, a pressure of 250 lb. per sq.in. is employed in all the soil samples. Sample 1, which consists primarily of a sand, is classed as an A-3 type of soil. Samples 3 and 5, which are clays, are classed as A-7 soil types.

Very little has been published on the theory of optimum moisture content for soil compaction, because of its recent discovery. Proctor (*ENR*, Aug. 31, 1933) believes that the optimum moisture content varies with the particle size. He states that the larger the proportion of fine particles present in a soil, the more water it will require to lubricate their larger total area sufficiently for the compaction to be most efficient. However, the opposite is found to be true for the compaction of the soil samples in Fig. 1. Sample 1 is a sand having a very low plasticity index and an optimum moisture content of 20.3 per cent, while samples 3 and 5 are two clays having a high plasticity index and an optimum moisture content of 17.4 and 18 per cent, respectively. From the hydrometer analyses in Fig. 3, which show the percentage of particle-size distribution of the different soil samples, it can be seen that the particles of the sand (sample 1) are much larger than those of the two clays (samples 3 and 5). The grain sizes of sample 10 are also larger than those of the two clays, but are smaller than those of the sand in sample 1. The optimum moisture content of sample 10 is 21.5 per cent, or 1.2 per cent greater than that of sample 1.

Since the grain sizes of sample 10 are smaller than those of sample 1, Proctor is correct in stating that it takes more water to lubricate the larger total surface area of the smaller particles. But if his statement were generally applicable, then the optimum moisture content for the clays should be greater than that of samples 1 and 10. However, this is not the case, and therefore his statement does not apply to soils in general but must be modified to include the effect of variations in soil





FIGS. 1 AND 2—Optimum moisture variations: (Fig. 1) with different classes and (Fig. 2) with different mixes of soils.

types. Sample 4, which is a Texas caliche, possesses the lowest optimum moisture content, but the particle sizes are greater than those of the two clays, and here again Proctor's statement would apply. The same is true for the two sands, samples 1 and 10. The reason for his statement applying to the caliche and clays and not to the sand and clays is still a problem to be solved.

However, the writer is inclined to believe that the colloidal content of soils is an important factor in this problem. Clays contain a large amount of colloidal material, while the two sands contain very little. The percentage of colloids that these samples contain may be determined from the hydrometer analyses in Fig. 3. When the colloidal particles become completely lubricated with water, they begin to swell. This causes the efficiency of the compaction to be decreased. Therefore, in clays the particles do not become completely lubricated at the optimum condition, while in the two sands and the caliche they do. It is true that the caliche also contains an appreciable amount of colloidal material, but it may not be present in the same state as that of the clays; nor is it likely that it possesses the same physical properties as the colloids of the clays. The caliche shows very little shrinkage tendency, whereas the clays show a very high shrinkage tendency.

Different mixes of soils

Fig. 2 represents the optimum moisture curves of the different soil mixes prepared in the laboratory. Sample 6 consists of a mixture of 56.7 per cent of sample 1 and 43.3 per cent of sample 5. This mixture is prepared by calculating the volume of voids in sample 1 when molded at its optimum moisture content for a pressure of 250 lb. per sq.in., and then adding a calculated amount of sample 5 to fill these voids as nearly as possible. The percentage of voids at the optimum condition or at any other point on the curve may be calculated by means of the following equation:

$$\frac{\text{specific gravity—apparent density}}{\text{specific gravity}} \times 100 = \text{percentage of voids.}$$

Results of the determination of the soil constants revealed that this mixture is an A-6-2 type of soil.

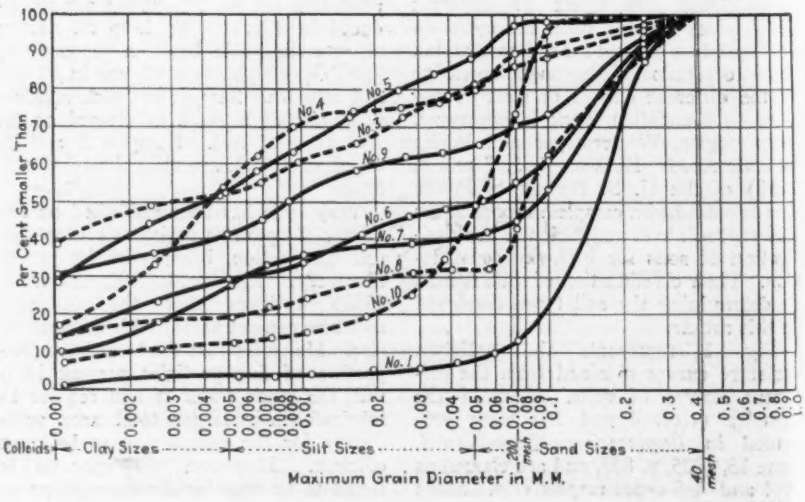
The material of sample 7 consists of a mixture of 56.7 per cent of sample 1 and 43.4 per cent of sample 4, which is a caliche. This mixture represents the A-2 type of soils.

Sample 8 consists of a plain mixture of 75 per cent of sample 1 and 25 per cent of sample 5. By a "plain mixture" it is meant that the optimum condition is not taken into consideration. Soil constants for this sample proved that it is an A-2 type.

The material of sample 9 consists of a mixture of 36 per cent of sample 1 and 64 per cent of sample 5. This mixture is classed as an A-6 type.

By a comparison of Figs. 1 and 2 it can be seen that all the mixtures possess a greater apparent density at the optimum condition than that of the original soil samples from which the mixtures were obtained. This is due to the fact that the voids in the sand that were previously filled with air and water are now filled with a more dense material such as clay and caliche. The greatest amount of compaction can be accomplished by taking the soil that possesses the greatest percentage of large grains and filling its voids with some finer material, such as clay or caliche. But the apparent density of a fine-grained soil

FIG. 3—HYDROMETER GRAIN SIZE COMPARISONS of soils, giving optimum moisture curves in Figs. 1 and 2.



cannot be increased to any great extent by endeavoring to fill its voids with a soil having a coarse texture. This principle is illustrated by samples 6 and 9.

Hydrometer analyses of soils

The method of procedure employed in the hydrometer analyses of the soil samples is according to the method first proposed by Bouyoucos (Bouyoucos: *J.Am.Soc.Agr.*, 23, No. 4, August, 1930) and later modified by Wintermeyer, Willis and Tooreen of the U. S. Bureau of Public Roads (*Public Roads*, 12, No. 8, October, 1931).

The results of the hydrometer analyses of the soil samples are presented in Fig. 3. The percentage of sand, silt, clay and colloidal sizes in each sample may be determined from its curve.

Although very little has been published on the subject of optimum moisture content for soil compaction, it will prove of value to the construction industry in the building of highways, foundations for buildings or the construction of earthfill dams.

Acknowledgments

Many helpful suggestions were obtained from Dr. E. P. Schoch, professor of chemical engineering in the University of Texas; A. D. Potter and Frank Newman, Jr., of the Texas State Highway Testing Laboratory, and Dr. H. L. Lochte, associate professor of chemistry in the University of Texas.

Concrete Arches Span 282 Ft. in Belgian Exposition Building

Over 2¾ acres of floor space unobstructed by columns is provided by construction utilizing three-hinged concrete arches—Base hinges are of ball-and-socket type of chrome-nickel steel

THE OPENING early in May of the Universal and International Exposition of Brussels in Belgium directs attention to the construction of the main building or Grand Palace, a reinforced-concrete structure of three-hinged arch type with no columns in an area 282x429 ft. The four other buildings of the exposition have interesting structural-steel framing and are of considerable size, but none of them approaches the main building in magnitude of dimensions or boldness of construction, according to an article in the magazine *L'Ossature Métallique*. Principal interest centers in the concrete arches and their construction on steel-arch centering.

FIG. 1—THREE-HINGED concrete arches of 282-ft. span being erected in the main building of the current Brussels International Exposition in Belgium. A group of two arches with their longitudinal bracing beams are shown being constructed on steel-arch centering in the background.

In its completed form the exterior of the building presents a series of terraces along its long dimension, and the arch-roof portion is adjoined front and rear by one-story annexes and tall entrance lobbies (Fig. 4), which bring the total length of the building to 524 ft. The terraces are built out from seven tiers of longitudinal beams resting on the arches, of which there are twelve spaced 39 ft. c. to c. The arch-roof portion of the building was constructed in six units comprising two arches each and the longitudinal beams, which are cantilevered a half panel beyond each arch. Expansion joints, entirely through the building, occur at the ends of these cantilevers, or at 78-ft. intervals.

The arch ribs have a uniform width of about 40 in. and vary in depth from 40 to 70 in. They are reinforced with

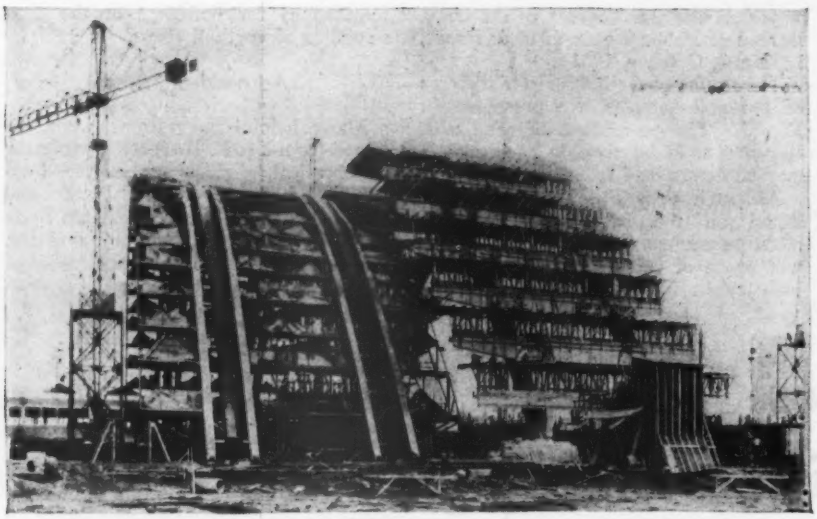


FIG. 2—REVOLVING CRANE assembling reinforcing steel for two of the concrete arches. The reinforcing was welded on the ground into units of 10 tons each. The building was concreted in sections 78 ft. long, one such completed section being shown at the right.

about 60 tons of reinforcing steel each, or 22 lb. per cu.ft. Intrados and extrados bars are 1⅛ in. in diameter; transverse bars are ¾ in. in diameter; and there are two rows of ⅝-in.-diameter spiral bars inclined at 45 deg. in the four faces of the ribs. These reinforcing-bar cages were fabricated on the ground by welding and were lifted into place in 10-ton units by a hammerhead-type crane moving on a track across the building site. The beam reinforcing was also fabricated by arc-welding.

The three hinges of each arch are made of steel castings, the crown hinge, transmitting a thrust of 400 tons, being of conventional type with a cylindrical pin. The base hinges, on the other hand, consist of large socket castings of chrome-nickel steel resting on a spherical ball of the same material 20 in. in diameter. They sustain an arch reaction of 900 tons. The base castings are mounted on concrete foundation blocks supported by 29 concrete piles, 25 of which are inclined 25 deg. to the vertical. These foundation blocks were made large enough to carry also the foot hinges of the steel-arch centering.

Each concrete rib was supported during construction by a pair of steel truss arches set 9 ft. 10 in. apart and joined by longitudinal trusses cantilevering about 15 ft. beyond each of the steel-arch trusses in order to support the forms for the longitudinal beams. As stated, a unit comprising two arch ribs was built at one time, so that two such sets of centering (four steel-arch trusses) were required. It was necessary to move this centering five times during the course of construction.

All concrete was placed (in metal forms) by means of concrete pumps capable of forcing concrete 650 ft. horizontally and 130 ft. vertically through pipes 5⅛ in. in diameter. Electric vibrators were used to assist placement of the relatively dry mix in the heavily reinforced members. Rapid-hardening cement was used.

Decentering was accomplished by raising the crown joint of the arch rib through the use of two 300-ton jacks. After the load was removed from the centering in this manner, a tie between the base hinges of the centering arches was tightened. The centering was then raised by means of jacks until the base pins could be removed, after which it was lowered onto a set of rollers operating on steel rails laid longitudinally of the building. The centering was then moved to position for the next group of arches.

Architect and engineer

The architect for the exposition is Joseph Van Neck, and the engineer Henri Demol. The Grand Palace was built by the Société Anonyme Engéma of Brussels.

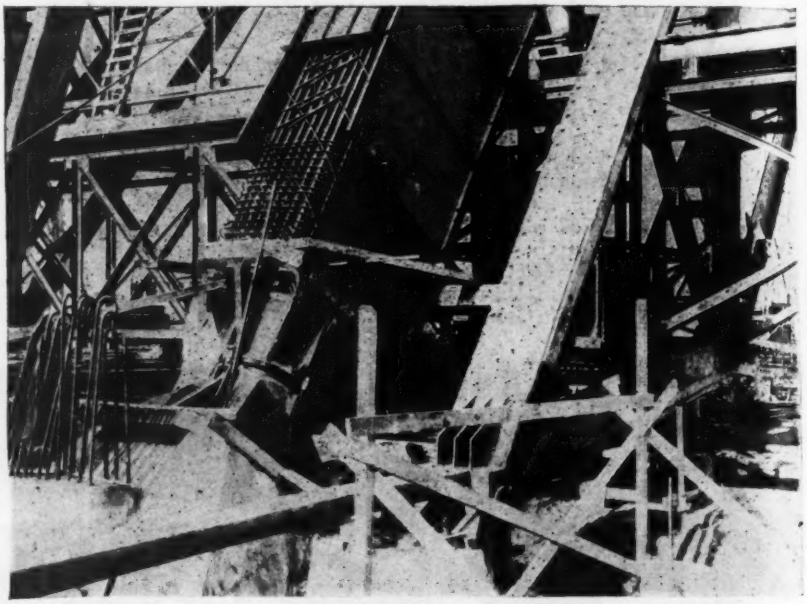


FIG. 3—BALL-AND-SOCKET HINGES are features of the heavily reinforced concrete ribs. Note that the steel-arch centering trusses, either side of the concrete rib are supported on an extension of the arch foundation.

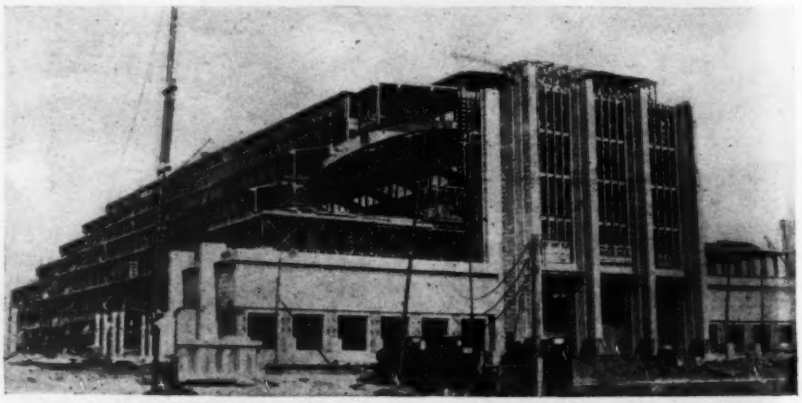


FIG. 4—GRAND PALACE of the Brussels International Exposition approaching completion.

Snow Surveys as an Aid to Flood Forecast and Control

Reservoir regulation in the interest of both flood control and water conservation facilitated by knowledge of snow concentration and melting rate—Information needed as to effect of warm winds in increasing the runoff

By J. E. Church

Meteorologist, Nevada Agricultural Experiment Station, Reno, Nevada

FLOOD PREVENTION and control now are becoming specialized divisions of snow surveying. Knowledge of snow depths and rates of melting gives the engineer greater accuracy in predicting the runoff of streams having their headwaters in mountainous regions than is possible from runoff data alone. Information so obtained is of special value both in the design of spillways for dams as well as in laying down a program of reservoir operation that will provide adequate area for flood storage without unnecessary sacrifice of stored water.

But the snow surveys already undertaken are far too meager to be of greatest value, and many unknown factors still limit the usefulness of the data that have been or may hereafter be collected. For example, a federal bureau recently inquired of this station as to the rate of floodflow to be expected from rapidly melting snow, especially under the influence of Chinook winds. Here is a subject that is of interest to a wide area in the western part of this country and of Canada, and yet it is one on which definite knowledge is almost completely lacking.

As indicative of fields into which snow-survey studies can be extended to advantage, the following comment is made on channel choking and rise in lake levels.

Channel choking

In the study of rapidity of melting and channel choking, little has been done, for our snow surveys have generally been limited to monthly surveys from January to June.

In every case flood effects must depend upon the elevation and the depth of snow. And frequently both of these may vary greatly within the same basin. For example, the Tahoe Basin, Nevada, varies from 6,225 to 10,900 ft. in elevation, the snow on the western half of its area is twice as deep as on its eastern, and the snow at its lower levels is probably less than one-fourth of that at its higher levels. Consequently, rapid melting is confined to its lower levels. Indeed, the only Chinook-effect I can remember began at not higher than 7,000 ft., and this on the lee side of the Sierra where the snow probably did not exceed 3 ft. in depth.

The rapidity of the runoff will also depend upon the condition of the soil, i.e. whether frozen, water-soaked or dry. However, frozen soil is unusual in the semi-arid West, where the soil drains practically dry each autumn.

The maximum menace of flood can best be determined by making snow surveys over the flood-producing area to ascertain the inches of water in suspension. By subtracting from this amount the amount necessary to prime the soil the net quantity of water in suspension will be made known. With these factors determined, the rate of release by melting will afford a basis for computing the channel deficiency.

Where a snow-survey system is in operation, the rate of melting and its effect on streamflow can be readily determined by sampling the snow field at the beginning and end of any melting period and gaging the resulting runoff. Thus a factor of relationship can in time be computed. However, we have not been able to find that the usual high temperatures can do more than accelerate melting and runoff. They evidently do not increase the total runoff indicated by the snow cover. In fact, Chinook winds are considered to be drying winds, removing the snow cover without equivalent runoff; and they certainly do accelerate evaporation to several times the normal rate.

Warm winds and rains combined are a very potent cause of floods, especially when acting upon a snow cover of several inches water content. The heavy flood in the Truckee River, March 22-28, 1928, was due to six days of rain, varying from 1.79 in. total at Reno (El. 4,500 ft.) to 10.17 in. at Tahoe City (El. 6,225 ft.) toward the upper end of the flood-producing area. The mean temperature at Tahoe City for the period was 35.9 deg. F., or 0.3 deg. F. below the mean for the entire month. And at Reno the mean temperature for the month was only 4.2 deg. F. above normal.

The snow cover had been light, and the monthly precipitation previous to the storm had been far below normal. The runoff rose from 359 to 12,000 sec.-ft. at Iceland, Calif., with a mean for the month of only 1,680 sec.-ft. as compared with a minimum of 905 sec.-ft., maximum 1,730 sec.-ft., and mean 1,220 sec.-ft. for the following month of April.

A snow-survey system with temperature and precipitation stations was organized many years ago for the Washington Water Power Co. (at Spokane) on Lake Coeur d'Alene, to determine the annual percentage of runoff in that basin and conditions tending toward flood. Since the watershed normally furnished far more runoff than could be stored in that lake and the lake could easily be badly flooded by leaving the outlet gates closed, the problem confronting the company was to determine when the seasonal runoff had so far diminished that the closing of the gates would raise the lake to the legal level and no higher. This study should have produced data of value, but as yet no detailed report has been made to me of its operation.

A comprehensive project, shared by the Nevada Agricultural Experiment Station and several state and federal bureaus, is now under way to study the relation of snow and rain to runoff in the Humboldt Basin, Nevada, a broad, alluvial basin with hampered channel. In this study some light may be thrown on the question of absorption and floods.

Reservoir reserve capacity

In the matter of a safety reserve in storage reservoirs that are maintained jointly for the use of irrigation, power or navigation and the prevention of floods, the problem is less involved; with adequate data it becomes one chiefly of management.

In this case the percentage system of snow surveying and forecasting streamflow meets every purpose.

This is particularly true west of the Continental Divide where the snow cover furnishes the preponderant portion of the annual runoff. It is necessary only to determine the total seasonal runoff from the snow cover and then to regulate the reservoir to meet probable requirements. If the gates of the reservoir are too small to provide quick regulation, the snow survey should be made at "key" stations throughout the winter, to assure early adjustment to impending conditions. Further, a safety area in the reservoir should be maintained to meet abnormal precipitation that may fall too late to be released from the reservoir in time to make way for the anticipated inflow.

This, in brief, is the system being presented to the government for approval by joint interests in regulating Lake Tahoe to avoid damage suits and still provide the maximum amount of upstream storage in the Truckee Basin. Thus, snow surveying, according to the Sierra Pacific Power Co., has become inevitable and furnishes the only way out of its present dilemma.

This method has been applied in re-

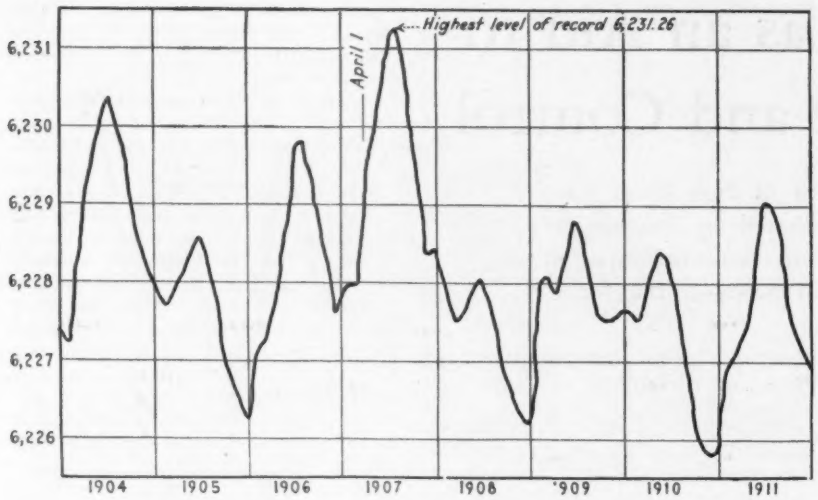


FIG. 1—FLUCTUATIONS IN THE LEVEL of Lake Tahoe 1904-1911, including the highest level of record.

trospect to two seasons of unusually heavy precipitation at the close of the accumulation period. The results, which follow, are convincing.

Rise of Lake Tahoe 1906-07

The season of 1906-07 is remembered as the year of the flood on Lake Tahoe, chiefly because the lake rose 1¼ ft. above its dam. This flood was probably quite minor as compared with that of the season of 1861-62, when a portion of the bank sloughed into the lake and the Sacramento and San Joaquin rivers were merged into one.

Since the normal annual rise (with gates closed) is 3.12 ft. and the normal rise after April 1, due mainly to melting snow, is 1.68 ft., it is obvious from the accompanying graph (Fig. 1) that the lake had attained maximum elevation in 1905-06 and was only acting within normality in 1906-07.

However, until March 1 of that year, it was possible that the precipitation would be so light that the maximum elevation would not appreciably exceed that of 1905-06. But a probably unprecedented precipitation of 20 5 in. at Truckee during March augmented the winter precipitation until it was double the normal, thus creating a situation in which the lake could have attained an elevation of 2.36 ft. above the level of the dam. On April 1, when the crisis was unmistakable, it would not have been possible, even with the present large outlet, to have drawn the lake down much more than 1 ft. during the three months of rise. Fortunately, the subsequent precipitation during this period was light, thus causing a deficiency of 0.5 ft. in the expected rise. Thus even despite nature's summer dryness, it is obvious that the lake's elevation was being maintained fully a foot too high. The detailed computation is shown in Tables I and II below. In studying these tables the following should be observed:

Excess above top of dam (2.36 ft.) = 288,000 acre-ft.
Drawable at elevation 6,227.09 = 72,000 monthly, (or for April-June, 216,000 acre-ft. = 0.96-ft. depth over the lake).

Excess beyond power to regulate = 1.40 ft.
Additional loss in expected rise due to lack of Apr.-July precipitation (approx.) = .5 ft.
Actual max. elevation, July 1 = 6,231.26 ft.

At the urging of the power company, snow surveys for forecasting Lake Tahoe rise were organized soon after this flood occurred and gave rise to the Nevada or percentage method of snow surveying.

Rise of Lake Tahoe 1927-28

The rise of Lake Tahoe in 1927-28 (Fig. 2) could cause no worry because of the low elevation of the lake then prevailing, though alarm was felt on the Truckee River because its channel was insufficient to carry the abnormal rainfall on its own watershed.

This abnormal rainfall was referred to in the discussion of channel choking. Nevertheless, the action of the lake indicated again the necessity of maintaining an emergency storage area in it to prevent flooding.

The March precipitation was abnormal as it had been in 1906-07, and followed a December-February precipitation of only 30.2 per cent of normal. Since the snow survey of April 1 was not made until after the storms of March, the record of precipitation at Tahoe is used. However, the snow survey was essential to show the amount of snow cover still available to raise the lake after April 1. The precipitations (in inches) are given below.

(Tahoe, El. 6,230 ft.)	Normal	1927–28		Normal	1927–28
Dec....	5.11	3.29	Apr...	2.09	3.63
Jan....	6.48	1.92	May..	1.11	0.10
Feb....	6.04	1.11	June..	0.49	0.28
Mar...	3.29	12.20	July...	0.31	0.19
Total..	20.92	18.52	Total..	4.00	4.20

The precipitation for both December-March and April-July was practically normal. However, the phases were extremely erratic, as shown in the following figures.

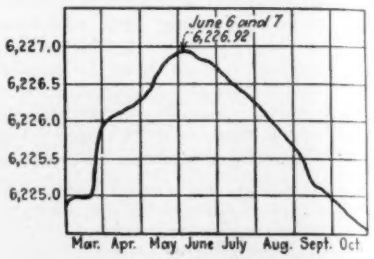


FIG. 2—EMERGENCY STORAGE area in Lake Tahoe, maintained as a result of snow surveys, could have kept the lake level within flood limits in June, 1928. However, the lake's low level automatically afforded requisite protection.

Per cent of normal precipitation, Dec.-Feb.	30.2
Dec.-March	88.5
Per cent of normal, snow survey of April 1	59.0
Excess temperature during March at Reno, deg. F.	4.2
Heavy rain March 22-28, in.	
Tahoe	10.17
Reno	1.79
Rise of Tahoe from effect of rain, ft.	1.03

The snow survey now became the only means of judging the further action of the lake. The levels and estimates based upon it were as shown in the tabulation on p. 881.

Had control been necessary, the following emergency might have been presented: The lake stood at El. 6,224.95 at the beginning of the March storm and at El. 6,225.71 at its close, and was still rapidly rising and so continued until it reached 6,225.98 ft. on April 5. This meant 128,000 acre-ft. to be discharged at the rate of about 72,000 acre-ft. monthly, or 216,000 acre-ft. for the three months before the

TABLE I—PRECIPITATION IN INCHES
(Truckee, El. 5,819 ft.)

	Normal	1906–07	Per Cent of Normal		Normal	1906–07	Per Cent of Normal
Dec..	4.09	5.10		Apr...	1.78	0	
Jan...	5.60	8.85		May..	1.07	0.80	
Feb..	4.94	2.30		June..	0.31	0.58	
Mar..	4.31	20.50		July...	0.18	0	
Total.	18.94	36.75	194.0	Total..	3.34	1.38	41.3*

*A similar correspondence is found also at Reno (El. 4,532 ft.)

TABLE II—FORECAST OF ELEVATIONS OF LAKE TAHOE
(Normal Annual Rise 3.12 Ft. Normal Rise, Apr. 1-July 1-7, 1.68 ft. Elevation Top of Dam 6,230 Ft.)

	Actual Elevation	Winter Precipitation Dec.-Feb. Per Cent	1906–07 Equivalent Rise in Lake, Ft.	Forecastable Max. Elevation (Gates Closed) July 1	Max. Reserve for Further Emergencies, Ft.
March 1.	6,228.0	85.8	1.44	6,229.44	0.56
April 1..	6,229.1	194.6	3.26	6,232.36	2.36

TABLE III—FORECAST OF ELEVATIONS OF LAKE TAHOE

	Actual Elevation	Winter Precipitation Dec.-Feb. Per Cent of Normal	Equivalent Rise in Lake, Ft.	Forecastable Max. Elevation (Gates Closed) July 1, Ft.	Max. Reserve for Further Emergency, Ft.
Mar. 1	6,224.81	30.2	0.51	6,225.32	4.68
Apr. 1	6,225.84	59.0*	0.99	6,226.83	3.17
Actual maximum elevation June 6-7				6,226.92 ft.	
Divergence from the forecast				+0.09 ft.	

*Per cent of the snow cover normally found on April 1.

crest would be reached. However, only by the use of this excess time could the lake have been controlled, for even on the monthly basis the discharge lagged 56,000 acre-ft., or 0.45 ft. behind the rise.

East of the Continental Divide

East of the Continental Divide the forecasting of floods will be more difficult because the snow cover furnishes the minor rather than major portion of the seasonal storage, and the effect of rains tends therefore to be immediate.

Traditionally, Chinook winds are drying, provided the snow is not too deep to be evaporated. However, the upper Missouri Basin should furnish good field for studying such effects. The Engineer Corps of the U. S. Army has now established a snow-survey system on seven watersheds tributary to the Fort Peck storage reservoir on the upper Missouri. Here is an unusual opportunity to analyze the causes of floods and build up a method of forecast. Increase in the detail of snow surveying and forecasting is the chief essential. This is also the land of Chinooks. Well-timed observations of evaporation and melting would quickly determine the power of Chinook winds.

Some laboratory work on snow-melting characteristics has been done by George D. Clyde (*Bulletin 231* (Technical), August, 1931, Utah Agricultural Experiment Station, Logan, Utah). A careful analysis of flood action under varying temperature has been made by J. H. Armington in connection with the Wabash-White River flood of January, 1930, in U. S. Weather Bureau *Climatological Data*, Indiana Section, January, 1930 (supplement).

Driving Sewer Tunnels Through Chicago's Blue Clay

Liner-plate ground support used in many variations—Power and hand knives peel off clay in strips—Many kinds of ground encountered

Second of a series of three articles.

CONTRACTORS on Chicago's 27-mile intercepting-sewer tunnel project, now under way, have developed tunneling methods peculiar to that city because of the character of ground encountered. While ground traversed ranges from a solid limestone through various clays to a fine running sand, mostly it is the characteristic Chicago blue clay of varying consistency. Steel liner plates of at least three designs have been skillfully adapted to varying soft-ground conditions. The use of needle beams and radial jack braces has been developed to a high degree of efficiency and safety.

The new sewage-disposal plan of the Sanitary District of Chicago was outlined in *ENR*, June 13, 1935, p. 831. It includes the construction of two new treatment plants, the extension of a third and the driving of 27 miles of intercepting-sewer tunnels ranging in internal size from 17½x19 ft. to 4x5 ft. This article is confined to the driving of the sewer tunnels tributary to the West Side and Calumet treatment works.

The map (Fig. 2) will orient the reader as to the location of the tunnels serving the West Side plant. Contracts 1 and 2, the first sections eastward from the plant, were completed in 1928 and 1930. The next two sections, contracts 3 and 4, 17x17 ft. in size, were started in 1930 but were shut down in December, 1931, when the Sanitary District ran into financial difficulties; work on them was resumed last year along with the start of the remainder of the interceptor program under PWA financing. A disastrous fire in the heading of contract 3 in 1931 caused the inauguration of many excellent safety measures now in effect, including the use of steel-plate lining instead of timber cants, formerly the standard on Chicago sewer tunnels.

Contracts 1 to 5, inclusive, run along the north and west sides of the South Branch of the Chicago River. At Taylor St., a branch to the east of contract 4 leads to the river bank. Here contract 6 begins with a double-barreled siphon under the river, thence following along the east side of the river up Franklin St. to Wacker Drive, turning eastward under the drive viaduct along the south bank of the main river to Michigan Ave.

Contracts 8 and 9 extend northward from Section 5 along the west side of the North Branch of the river and a part of contract 9 crosses to the east bank through a siphon and thence north to Fullerton Ave. Taking off at the junction of contracts 5 and 8, contract 7 crosses the North Branch in a three-barrel siphon and continues eastward in Kinzie St., paralleling the main river, to the outer drive, some distance east of Michigan Ave. at the Lake Shore line. The last section of the West Side system, contract 10, takes off from contract 7 just east of the river siphon and runs northward along the east side of North Branch.

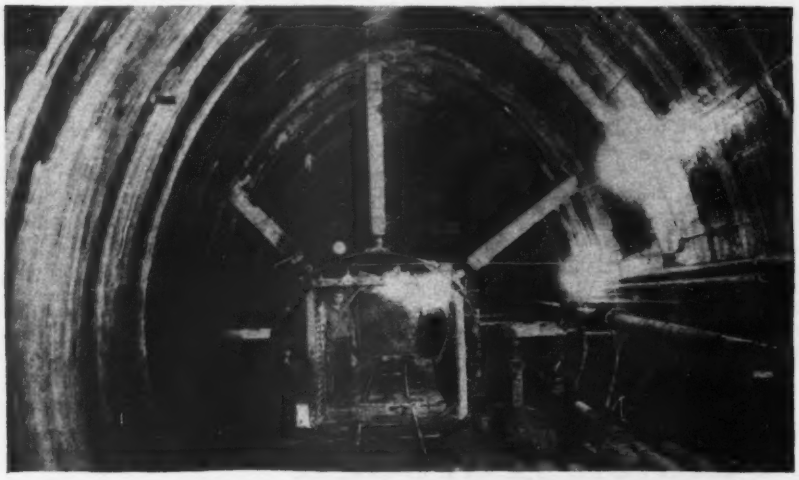
This system of interceptors serves the very heart of the city, including the Loop District, between 12th St. and the Chicago River on the south, to Fullerton Ave. on the north. In this district is found every kind of ground known to Chicago territory. Much grief was caused contractors by forgotten and long filled up slips leading from the river, with old timber débris and even a drydock being encountered in tunnel driving. Temporary falsework for the Wacker Drive Viaduct was also encountered, adding to driving difficulties on contract 6.

Six miles of interceptors and connecting conduits were driven in tunnel for the Calumet plant at 125th and Michigan Ave. (not shown on the map). Soil varied from a hard blue clay that required no support to soft mixed materials that required full steel lining and 12 to 16 lb. of air pressure.

Compressed air required

Except for the all-rock section on contract 3, part of the outlying small tunnel on contract 9 and the connection tunnel at Calumet, compressed air from 5 to 9 lb. pressure is required in all headings. Because of the small size of many of the sections, several interesting arrangements have been developed for air locks. Single locks are used for both men and materials, as space limitations do not permit of separate man locks usually found on larger tunnels. On contracts 3 and 4 (17-ft. bores), regular 6-ft. cylindrical locks, 40 ft. long, are used, installed through a single concrete bulkhead built into the concrete tunnel lining. To conserve air

lost by locking out, the contractor on Section 6 (9 ft. 4 in.x11 ft. 8 in.) devised a 6-ft. cylinder out of regular tunnel liner plates bolted tight without any stiffening ribs, set between two concrete bulkheads equipped with steel doors. The space between the outside of the liner plate and the inside of the regular tunnel section was then backfilled tight with puddled clay to provide support for the plates. Fig. 1 shows three types of locks.



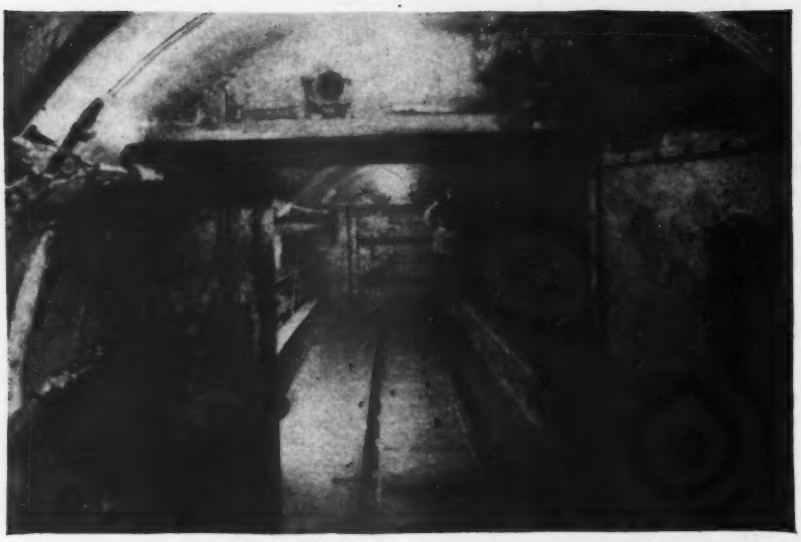


FIG. 1—THREE TYPES OF AIR LOCKS in use on the Chicago sewer tunnels. Top—conventional 6-ft. cylindrical lock in 17-ft. tunnel; center—steel bulkheads built into tunnel lining form the lock on an access tunnel; bottom—steel liner plates make up the lock chamber, and concrete bulkheads form the ends; the space between the lock and tunnel walls is backfilled with puddled clay.

On the smaller sections the locks are merely steel or concrete bulkheads equipped with steel doors, with a short section of complete tunnel serving as the lock chamber.

Emergency locks are designed to accommodate at least 40 men at once and are equipped with three doors, to permit escape from the tunnel or access to the tunnel by rescue crews. Three types of emergency locks are used. On contract 3 the lock chamber is a steel cylinder carried on a high scaffold within the tunnel, with a free-air horizontal connection to the foot of an emergency shaft. The usual emergency lock consists of a vertical shaft just off the tunnel line but connected with the tunnel by a direct opening. This shaft leads to a horizontal lock on the street surface, equipped with three doors. A third type, preferred by the engineers because of the desirable feature of enabling the men to get out of the air at the tunnel level, is a horizontal chamber built expressly for the purpose, or else installed in a side-connecting sewer. This chamber is equipped with a lock door on the tunnel side. A vertical shaft rises from the roof of the lock chamber, equipped with two drop-flap gates, one at the bottom, the other at the surface. The swinging door and the top-flap gate are kept open; the bottom-flap gate is held closed by air pressure. All doors and gates are fitted with proper valves for manipulation of air pressure. Thus these emergency locks can be entered from either side, from within or without the tunnel.

General methods

In soft ground the tunneling methods on all sections follow a somewhat standard procedure, though naturally in a range of sizes from that large enough for a railroad tunnel down to cramped little 4x5-ft. holes, construction methods must, by necessity, vary. Then, too, each contractor prefers a particular procedure, the comparison and contrast providing an interesting study of tunneling operations.

Excavation of the peculiar Chicago blue clay is done by a method seldom seen elsewhere. This material cannot be handled by any kind of a mucking or excavating machine but must be sliced off faces and benches by U-shaped knives operated by hand or a hoist line. While some of this clay will stand unsupported, especially in the smaller bores, most of it requires instant support. Pressed-steel liner plates are specified for such support where needed. Sometimes the steel plates erected between structural-steel ribs will support the roof and sides alone, but usually a needle-beam arrangement with radial jack braces is necessary to keep the

plate lining from collapsing. The extreme in shoring and bracing is reached in free-running sand, where every square foot of sides, roof and heading face must be sheeted. Careful breasting of the face is required in such cases, and the needle-beam bracing is supplemented by drums or cross-tunnel bracing on the lower side walls.

In brief, the driving of all soft-ground tunnels involves first, excavation, followed by immediate support of the ground by the liner plates, the headings being carried ahead only the distance that they can be mined out and lined with permanent concrete structure in one day's (24 hours) work. After excavating, the reinforcing is set, concrete invert is placed, the walls and arch are poured, and finally, following a day or so behind the concrete operations, the arch is thoroughly grouted, to fill any voids between the concrete and liner plates and outside the rings of plates. The last step in the tunneling operation is to place a thin concrete finish on the curved invert.

Mining methods

Because of cramped working space in the smaller bores, the use of needle beams is restricted to tunnels larger than 7 ft. in diameter. In the larger bores the first operation is to advance a pilot drift, from 2x4 to 3x5 ft. in size, about 30 ft. ahead of the face. This drift, called a monkey hole, often requires some sort of bracing and support, and is driven solely to receive the advancing needle beam. These beams are two 12-in. or 10-in. I-beams, 30 ft. long, bolted together with a square timber filler (Fig. 4).

The front end of the needle beam is blocked up on the bottom of the monkey hole; the rear end is carried by a post set on the invert of the previously placed concrete section. Advancing of the monkey hole usually occurs during the concreting operation of the previous section, enabling the mining crew to start the main excavation by the time concreting is finished.

Once the needle beam is set, the first mining crew advances a top heading, mining out with clay knives or air spades where the ground is too hard for the knives. As the top heading progresses, liner plates and their supporting ribs are installed. Every 3 or 4 ft. a group of five or seven radial jacks is set up braced against the lower flanges or the top of the needle beam and bearing against short timber wales placed next to the plates and ribs. The vertical brace, resting on the top of the needle, is usually a timber strut fitted with a screw jack; the side braces are usually the common sewer-trench pipe jack.

After the top heading has been carried through to the end of the monkey hole, another mining crew excavates the bench to grade. A bench about 6 ft. long is left in place against the face to receive the muck from the next advance of the monkey hole.

FIG. 2—ROUTE of the West Side intercepting sewers in Chicago, which serve the congested Loop business district and near-by areas.

Mining the clay

Excavation is largely a hand operation. In the top heading, hand knives are the main tools. The face of the heading is worked vertically usually with two men drawing the knife, a U-shaped blade with two handles, from top to bottom, shaving off a vertical slice. As the slice or slab of clay is cut loose, two muckers stand by to grab it and cast it back to more muckers, who rehandle it into waiting cars. Towards the end of the cut, the muck is often rehandled three or four times. The clay is sticky, yet firm enough to hold together in these long slabs as cut from the face.

There are two methods in common use for mining out the bench. Here it is possible to get a horizontal, longitudinal pull on the clay knives and, when the consistency of the clay permits, the knives are pulled from the face back toward the tunnel by a line from an air hoist. In this operation one man guides the knife, a U-shaped cutter with a hoop handle (Fig. 8). As the slice is cut loose, muckers with sharp spades cut the slab into chunks for loading by hand. This is back-breaking work, as

FIG. 3—GROUND CONDITIONS vary widely on the West Side sewers. Left: Typical blue clay under Van Buren St., firm enough to stand unsupported. Center: steel-plate lining on sides and top in softer clay. Right: Full plate lining and needle-beam bracing required at the east end of contract 7. Joints are plastered with mud to hold the compressed air.

the clay weighs about 165 lb. per cu. ft. Power knives can be used only where the clay is of the right consistency. If it is too sticky, the cut made by the knife will instantly close up. In such ground hand knives must be used and must be worked from a vertical face, so that the muckers can pull off the slice as it is cut loose.

On one contract an unusually fine clay formation led to an interesting development of the use of the power knife. Here a needle beam was unnecessary, as the ground stood alone until the full-size heading was mined out for a distance of 12 ft. or so, after which plates and ribs were installed on the arch and part way down the sides. A monkey hole was advanced in short sections at the bottom center of the heading. Then with the use of a power knife, slabs were cut off the sides and top of the monkey hole, gradually widening the hole to the full size of the heading. This is the most ideal use of the power knife found on the entire project. Occasionally a contractor would use the power knife for advancing the monkey hole, but seldom was the ground found to be of the right consistency for such work.

On the bores too small for the use of a needle beam, a full-face heading is carried, mined out with hand knives, with the muck car being kept close to the face on temporary track. In all headings where ground too stiff for cutting with a clay knife is encountered, pneumatic spades are used to good advantage.

Steel-plate shoring

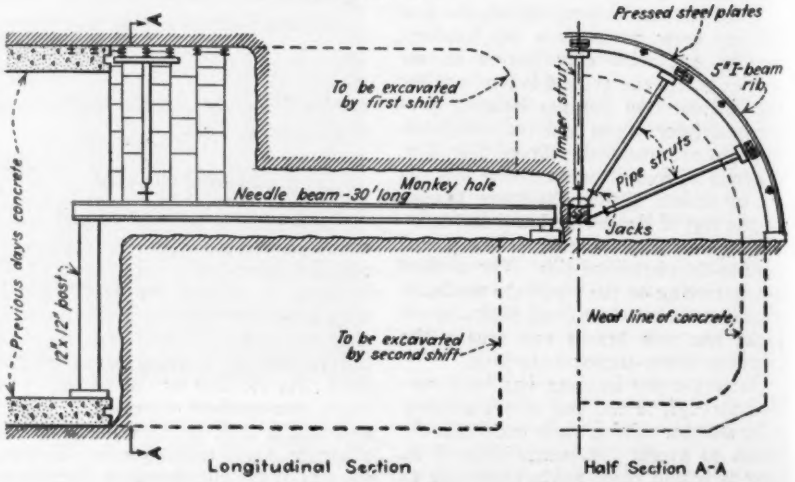
Contractors are using three types of pressed-steel plates, all in connection with I-beam ribs. One is the Commercial plate, 16x37 in., of ⅛ or ⅜-in. metal corrugated and flanged on all sides. Another is the Chicago Tunnel Sheathing plate, 24 in. wide of corrugated plates, with 2x1½x⅜-in. cross-angles, spot-welded across the corrugations and fitted with a short return section for bolting to the ribs. A third type, the Youngstown plate, is 12 or 24 in. wide, corrugated and flanged all around, with cut corners leaving a diamond-shape opening at the intersection of four plates. Grout pipes are inserted in these openings; on the other plates the grout-nipples are inserted through holes punched in the pans for that purpose.

The steel ribs, of 3, 4 or 5-in. I-beams, are jointed, to permit partial erection, with provisions for plate splicing. Various arrangements of ribs and plates are used in the several contracts. Sometimes the ribs are carried to benches cut into the side walls at various elevations; again they are carried down to subgrade (Fig. 3).

Plates are always installed in the crown of the tunnel, but the extent to which they are carried down the sides depends upon the stability of the ground. Often plates are carried above springline only; again the entire side wall and arch perimeter will be protected. On the 9-ft. connecting tunnel at the Calumet plant not only was the usual needle beam dispensed with, but a set of two or four ribs with connecting plates at the crown only was required only every 8 ft. or so. In this case the ribs stopped at springline (Fig. 5).

The other extreme was reached in the south heading of contract 4, a 17x17-ft. section. Here a stretch of running sand was encountered, requiring not only full-side and arch shoring but a careful breasting at the face of the top heading and a bulkhead of timber sheeting across the face of the bench. In this bad ground the liner plates and ribs were carried above springline only, with a

FIG. 4—STEEL-PLATE LINING with needle-beam bracing is the typical temporary support used on many tunnel sections. The needle is advanced in a small drift or monkey hole.



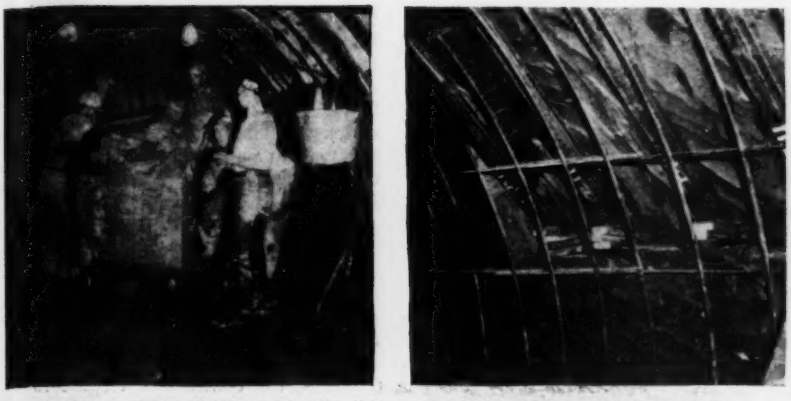


FIG. 5—FIRM CLAY requiring but little support is found in some headings. Left: A heading under Wacker Drive that needs only roof support of steel plates with occasional ribs carried to invert. Right: Clay trimmed to exact profile by air spades at the Calumet connecting conduit, with an occasional set of ribs carried on a side-wall bench and supporting a crown plate or two.

FIG. 6—GROUND SUPPORT in a 17-ft. tunnel in fairly good clay consisted of self-supporting ribs and plates carried on a side-wall bench, with the lower side walls protected by wood sheeting braced by cross-drums. This is a view in contract 3.

row of vertical wakefield sheeting being driven for the support of the lower side walls. This sheeting was braced by cross-struts or drums passing beneath the needle beam. Excavation in the heading was slow and laborious. In the face and on the bench one board only could be removed at a time, and a small amount of sand taken out before the board had to be replaced. Fortunately for the contractor the sand formation ran out after a few hundred feet, and excellent clay was encountered.

Another difficult stretch of ground is on the east heading of contract 7, where the tunnel reaches out into the thoroughly saturated Lake Michigan white sand. This is in the now-famous Streeterville section beyond the original beach built up by natural accretion of sand. This ground will not hold air, and to traverse it at all it is necessary to seal over the joints of the full-section steel-plate lining with soft clay from another heading. The face of the heading will stand without breasting if it is plastered with clay to prevent loss of air. Naturally nothing can be done with the sand bottom and considerable air escapes here, bubbling up through the wet sand around the lining shell with a disquieting gurgle.

As the tunnels reduce in size toward the outlying ends of the system, mining becomes more difficult because of the small working space. Miniature cars are used on 14-in.-gage tracks. Men walk and work stooped over. Yet good progress is maintained on these small bores.

On the parts of the east heading of contract 3,, a large section, there was some variation from the usual procedure where the earth was so firm that it was not necessary to use a needle beam. Tunnel driving was carried on in a temporary lining of steel plates bolted together, and each ring of plates was supported by a 5-in. I-beam rib bent to correct shape. The ribs were brought to a firm bearing on sills or blocks on a bench in the side walls just below the springing line of the arch. The first operation was to take out a cut 3 ft. wide to a depth of 16 in. across the full-arch portion and place one course of liner plates and a rib. Then a cut was made for another course and so on until a 20-ft. length was mined, leaving a prism of undisturbed earth inside to be mined out in the next operation. As the mining progresses the muck cars were moved up to the face on a temporary track and when loaded were pulled back onto the working floor by a portable air hoist. Occasionally where the earth became softer than usual, it was necessary to support the side walls below the steel lining with planks and drum braces extending across the tunnel.

In driving a small drift to contain an emergency air lock, one contractor employed the familiar Chicago well method, of 2x6-in. lagging held by split steel rings as braces; only here the lagging was, of course, placed horizontally.

Rock work

Contract 3 is notable on the West Side system for containing the only rock on the entire 27 miles now under construction. For 5,900 ft. rock is found in the lower part of the bore, clay in the upper part, with the rock line running all the way from invert to roof. Much of the time it skimmed the tunnel roof, a most difficult driving condition. For the last 500 ft. on the west end the heading was entirely in rock.

At first, when the rock line was below the center of the tunnel, a heading

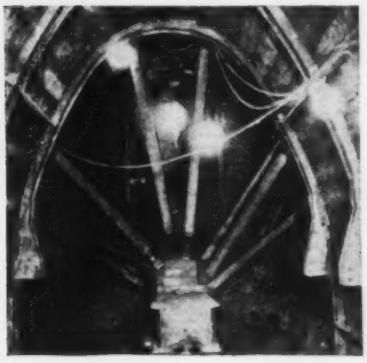


FIG. 7—TYPICAL needle-beam set-up as excavation of a new advance in one of the Chicago tunnels gets under way.

FIG. 8—POWER KNIVES pulled by air hoists slice through the clay if the consistency is right. These hoop-shaped cutters cut a thick slab of clay loose, and workmen cut the slab into chunks with spades for hand loading.

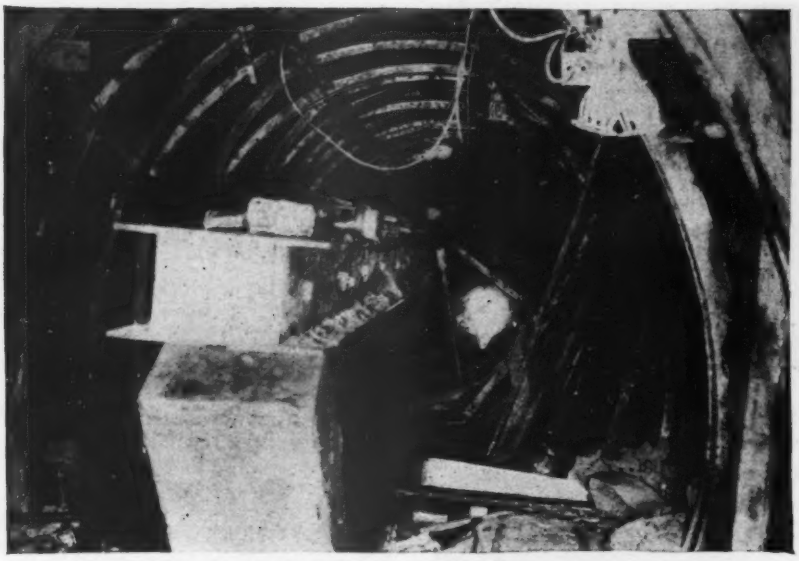


FIG. 9—ALL IS NOT WELL all the time. Here the liner plates and supporting needle-beam system gave way in ground saturated by a leaking water main overhead. This section was being driven in free air.

FIG. 10—MECHANICAL LOADERS are scarce on the tunnel system, for there is no satisfactory way to handle the sticky clay except by hand. Contract 3 traverses a rocky reef in the clay where this air mucking machine is useful.

through the clay was advanced for 8 or 10 ft. at a time. Steel-plate lining was installed with the ribs carried either to a clay bench or to rock, depending upon the height of the latter. Then the rock face was drilled with jackhammers and blasted out. This scheme worked well until the rock line approached the roof. Then blasting was found to disturb subsurface utilities.

Tunneling operations were changed to driving a 10x14-ft pilot drift at the bottom of the tunnel entirely through the rock section. This pilot was driven under free air from the main face and both ways from a second shaft sunk for that purpose. Drifter drills mounted on columnns and bars were used to good advantage, a full-face heading about 8 ft. deep with a square center cut being pulled each round. Muck was handled by a small loader, the only mechanical loader on the entire tunnel system (Fig. 10).

Once the pilot drift was completed, driving of the face was resumed under air, breaking down the rock into the drift by jackhammer drilling. The top heading in clay was advanced along with the rock face, though at times working space became cramped. Because of the downward relief of blasting pressure, no more trouble was experienced in the overlying utilities.

Mechanical haulage is used in contract 3, contract 5 and from the Quincy St. shaft of contract 6, using storage-battery locomotives on 30-in.-gage track. Safety restrictions placed all battery-charging equipment on the surface. Mules do much of the hauling on other contracts, handling from four to eight cars at a time. New specifications prohibit the use of animals in the tunnel, so this method of locomotion will soon pass out. On contract 10, where tunnel sections are small, Shetland ponies are used, for mules would only add to the already cramped working conditions. All of the animals change shifts with the men, and do not seem to mind the trips through the airlocks.

On one heading each in contracts 6 and 9, the contractors elected to do their own haulage with man power. Passing tracks are installed every 700 or 800 ft., and one man is responsible for moving all cars, one at a time, between passing points.

Concrete-lining methods and general plant layouts will be described in a succeeding article. The previous article of this series gave the personnel of the supervisory engineering forces and the contractors.

Canadians to Supervise Surveys in Newfoundland

THE Dominion Government of Canada, at the request of the Commission of Government that is at present administering the affairs of the British colony of Newfoundland, has consented to assist in the laying down of a basis for the development of survey work in the island, Canadian government survey officers being made available for this purpose. The Geodetic Survey of Canada will cooperate with officials of the island government in establishing two main nets of triangulation in Newfoundland. Work is scheduled to start this summer, and will probably include an aerial reconnaissance of the whole net by methods used with great success in hundreds of miles of Canadian triangulation. Under the financial arrangements the Newfoundland Commission of Government will bear the chief cost of the work, for which the sum of $75,000 has already been appropriated.

Canada has a direct interest in this work since it will complete a circuit of 1,300 miles of triangulation around the Gulf of the St. Lawrence, about 1,000 miles of which is in Canadian territory. Apart from this value, the triangulation net will have an added importance since the Canadian system along the north shore of the gulf passes within a comparatively short distance of the Quebec-Labrador boundary. The net must therefore eventually be the foundation for any future action between the Canadian and the Newfoundland governments over this boundary, action which would seem to be inevitable in view of the importance of the territory.

Letters to the Editor

Safer Bridges

Sir—An editorial in your issue of March 14, 1935, calls attention to the frequent crashing of motor vehicles into bridge railings of insufficient strength to resist the crash, and the serious accidents caused thereby.

Either the roadway curb or the bridge railing, or both, may be designed to furnish the necessary protection. In order to be able to use the curb as a protection, it must be of sufficient height and adequately supported to resist horizontal as well as vertical loads. In streets (and some bridges) the curb height is made from 5 to 8 in. as a convenience for pedestrians. On bridges, however, it is unnecessary to maintain the low curb on that account, except possibly at the ends of the bridge. If, then, the curb height is made from 14 to 16 in. or more, the curb will make an efficacious barrier against a vehicle attempting to mount the sidewalk. If ice, snow or other matter is liable to accumulate on the roadway at the curb line, the greater depth is advisable, because such accumulations reduce the effective depth of the curb.

It is true that a high curb has a tendency to keep vehicles from driving close to the curb line, causing part of the roadway width to be wasted, but with the present trend of making the width of the roadway lane adjacent to the curb not less than 11 or 12 ft. there will be little if any roadway width wasted by reason of the high curb. Extra cost is more than repaid by the additional traffic safety attained.

In addition to a high curb, the sidewalk, or any other part of the bridge structure that backs up the curb, must be strong enough to withstand the collision force. This can generally be arranged by designing the sidewalk structure as a horizontal girder of the proper carrying capacity and anchoring it in a suitable manner to the main structure of the bridge. It will often be found that the sidewalk structure designed without reference to this condition will be ample to resist the impact of the collision.

Whether or not the railing should be designed as a protection for vehicles colliding against the bridge structure depends on the height of the curb and the width of the sidewalk. If the sidewalk is narrow and the curb is low, the railing should be strong enough and sufficiently well anchored to withstand the full impact. If the sidewalk is wide and the curb reasonably high, a somewhat lighter railing may be proper, but caution should be used in reducing its strength, because a combination of unexpected conditions may cause the railing to receive the full blow. If the curb is so high that the wheels of the vehicle cannot mount the sidewalk, and the sidewalk is so wide that the detached body of the vehicle cannot reach the railing, the latter may be designed without reference to the colliding vehicle.

In addition to having the necessary strength, it is often advisable that the railing have the appearance of protective strength. A good many people have a fear of driving at great height or across water when close to an unsubstantial-looking railing. This fear is likely to translate itself into uncertain driving, which may cause an accident on the roadway. If may often be proper, therefore, to provide a substantial-looking railing, high enough to make the driver feel secure and of sufficiently close pattern that the driver will not be frightened by seeing a large expanse of water or earth far below the bridge.

For several years the New Jersey state highway department used the concrete railing shown in Fig. 1, and the records indicate that, although many collisions have occurred between these railings and motor vehicles of all types and sizes, in no case has the railing given way to such an extent that the colliding vehicle has fallen off the bridge. In practically all cases the damage to the railing has been slight. From this experience it may be presumed that this type of railing is sufficient for the purpose.

Whether the railing is unnecessarily strong is more difficult to say. It is known that certain concrete railings, 6 in. wide and with lighter reinforcement, have not been able to withstand a collision. It is probable, therefore, that no material reduction in the strength of the type shown is advisable where the sidewalk is narrow. For bridges with wider sidewalks there has been used a similar type of railing with widths of posts and panels of 75 per cent of those shown, but with the same amount of reinforcement. These railings have also been found satisfactory.

Similar substantial railings can be made of steel, the details depending on the design of the bridge and the idiosyncrasies of the designer. In Fig. 2 is shown the design of the steel railing, sidewalk and curb used on the high-level viaduct between Jersey City and Newark.

SIGVALD JOHANNESSON,
Designing Engineer,
New Jersey State Highway Commission.
Trenton, N. J.,
April 10, 1934.

FIG. 1—CONCRETE RAILING that has proved strong enough to save cars and trucks from falling off New Jersey bridges.

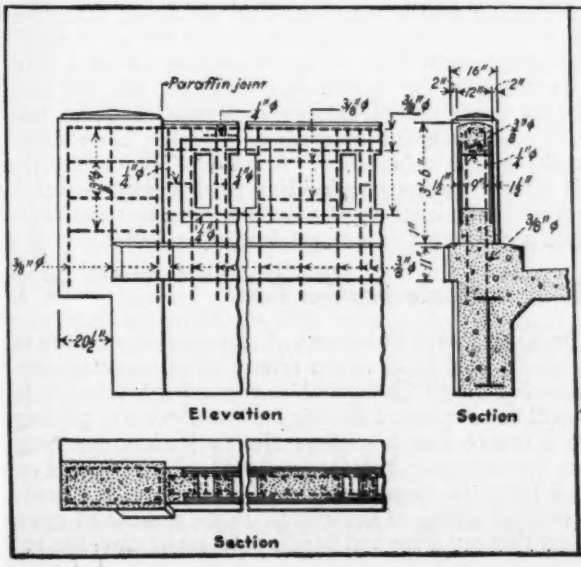
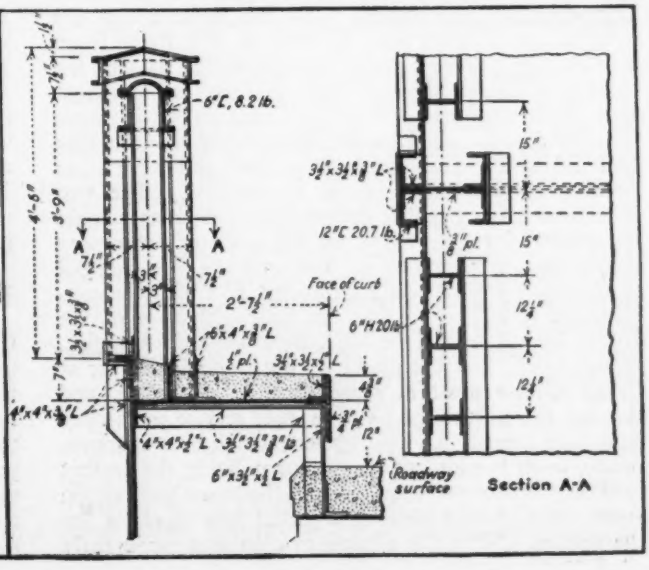


FIG. 2—RAILING, SIDEWALK and curb design used on the high-level viaduct west of the Holland Tunnel.



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June 20, 1935

Engineering in Human Service

SOME of the implications of the assembly of engineers that met last Thursday to honor Col. William J. Wilgus for his service as New York relief-works administrator go to the heart of questions that deeply affect the engineering profession today. It should be recorded first of all that the occasion was marked by a rare spirit of appreciation of self-sacrificing public service. It expressed a consciousness of the duty to vindicate one who had been wronged in its performance —for all present knew that Colonel Wilgus' devotion had earned the common reward of official ingratitude tinged with political motive and self-seeking. It was gratifying particularly to find mechanical and electrical engineers joining with their civil colleagues in giving voice to this spirit. But the meeting was perhaps most significant in symbolizing the function of the engineer in today's public emergency service and in revealing the fact that this function is widely recognized. The speakers at the assembly unmistakably expressed the obligation which rests on the profession to take up its share of the burden of such service. In doing so they gave recognition to a truth well proved by current experiences in the tangled administration of relief and recovery activities. Such acknowledgment of the obligation to place engineering skill in the service of great human problems is evidence of a force that will carry the profession to greater heights.

Grand Coulee Revised

UNDER THE CHANGE in construction plans announced a week ago, the Grand Coulee project is fundamentally altered. What heretofore was an engineering and economic error is transformed into a sound and justifiable work-creating undertaking. As we pointed out six months ago (*ENR*, Jan. 3, 1934, p. 23) there is no warrant for a present power development at the site, the more so while the initial market to be furnished by irrigation pumping demand for the Columbia Basin lands remains absent. And on the engineering side the plan to build a low dam and later thicken and raise it to form a high dam is subject to question. The two objections taken together outweigh any merit that could be claimed for the project on the score of emergency employment produced by it. But with the plan changed to eliminate present power provision, and with the construction replanned so that the full-width base of the future high dam will be built, these objections vanish. Granted only that the power of the Columbia River at Grand Coulee must at some time be developed, and that for this purpose and for possible service as forebay of a large irrigation supply system a high dam is appropriate, there is excellent reason for utilizing the present unemployment emergency to build the lower part of the future dam, thereby creating a partial flow regulator for the stream. Since the change in plan was made early enough to involve no waste of work already done, the decision reached appears in all respects commendable.

Equity Is Good Law

WHENEVER the intricacies of contract law as it affects engineer and contractor seem to create a new problem it is well to remember the old rule that equity between the parties to a contract governs its interpretation. What they understood in their meeting of minds, or what they should and reasonably might have understood, defines the contract obligation. This principle is called to mind by a court decision reported last week (*ENR*, June 13, 1935, p. 862), where certain stipulations as to delay in completion had been made but where the actual delay caused to the one contractor by the other exceeded anything that could have been in the minds of the parties. Literal reading of the delay condition might have imposed a limitless obligation; but to impose an obligation that may become unlimited, under a contract clearly intended to provide for limited contingencies, would be out of accord with equity; and since neither owner nor contractor would deliberately enter such a contract, the delay clause evidently must be so interpreted as to confine it to reasonable limits. The court decision, by taking this view of the contract, confirms the belief that equity is good law in contract questions.

Stage Roadbuilding

STAGE CONSTRUCTION OF ROADS assumes new significance in the light of the recent orders from Washington to limit relief-work expenditures so narrowly as to permit of nothing beyond grading. Not many years ago stage construction was being extensively argued as a fundamental policy of highway improvement. It was even spoken of as something new and distinctive in highway practice, although virtually all road engineers were using it on occasion. But it was not then nor is it yet a widely accepted plan of road improvement. Wherever he could the engineer planned and undertook grading and drainage and permanent surfacing as a single continuous operation. This practice, as was stated a week ago, is not barred by the present federal financing set-up, if relief and federal-aid funds are combined purposefully. On the other hand, if work must be confined in part to grading and to temporary light surfacing, waste need not result; a fundamental element of the main road is provided ready for the superstructure of such better surfacing as the future may demand. On the older paved roads the $1,400 per man-year of direct labor will cover widening of grade and ditches, bank sloping and shoulder stabilization. A splendid opportunity thus is offered highway engineering to demonstrate that by its skill and management relief work can be transformed into a permanent investment for the road user.

Where Modernization Fails

CHARACTERISTIC PRACTICES of clay excavation mark the methods used in driving a system of intercepting-sewer tunnels through Chicago's blue clay subsoil. Except for resort to compressed air, the basic methods are primitive to a degree found nowhere else in modern tunneling; yet the enterprise includes nearly 30 miles of tunnel and will form the collection works for the greatest sewage-treatment system of the world. In the absence of equipment that can mine and handle soft, sticky clay, the tun-

nels must be advanced mainly by slicing off the clay a sliver at a time by hand knives. Pressed-steel liner plates for supporting the roof afford the one modern touch. Occasionally local ground conditions permit the adoption of a power clay knife and sometimes air spades, but even then the heavy material must be handled and loaded by back-breaking hand work. With driving dependent upon hand operations, haulage and hoisting naturally fail to follow improved methods, and mule or hand tramming and open-shaft hoisting are common. Similarly, elaborate concrete-placing methods and form systems have no place in these small tunnels. The interesting tunneling procedure is dealt with in detail in three articles, the second of which appears in this issue. They give an instructive picture of methods which though simple are precisely fitted to the physical conditions encountered.

Much Work to Be Done

THAT PROBLEMS of many kinds and of great magnitude confront the code-free construction industry was made clearly evident by the discussions at the Construction League meeting at Washington this week. The meeting had the primary purpose of determining what can be done to consolidate gains made under the Recovery Act, but the problem contained in this simple phrase is not one to be solved overnight. It is by no means merely a matter of maintaining the fair-practice provisions of the code, for codes were highly imperfect compromises.

It is essential, therefore, to think of the fundamental needs of industrial regulation and advancement rather than of what the codes contain. While this implies slow progress, yet the urgency of many of the problems involved may not be forgotten. Trade practices may degenerate quickly, sound labor relations are equally of immediate concern; in addition inter-industry questions, statistics and planning for stabilization are but little less urgent. Means for dealing with these problems through cooperation of the groups comprised within construction demand early study.

Two steps of progress are to be recorded, the proposal of the Commerce Department to set up an industrial service unit for the construction industry, and change of the League's organization that will permit of retaining the local coordinating committees created under the auspices of the Construction Code Authority. These are the beginnings of progress toward self-management of the industry. A great amount of work remains to be done.

Officially Confirmed

WHAT the President said to the state relief administrators in addressing them at Washington on Monday of this week definitely confirms the belief that the $4,000,000,000 relief-works program is destined to be a repetition of the CWA. The President reaffirmed the cost limit of $1,100 per man-year of direct labor, and pointed out that many fine, useful projects will have to be rejected in order to meet it. All consideration of employment created in shop, mine or mill to supply the material for a project is abandoned. As we said a week ago this means the elimination of normal construction work, including PWA projects and the major part of federal construction.

Public-works engineers are thus placed in a new situation, with twofold responsibility. Manifestly they should give all possible aid to the works program. But also they must turn to considering the improvement needs of their own communities.

It becomes their first duty, then, to discover and develop the maximum number of sound projects of direct-labor type available for the federal works program. In accordance with the specifications laid down by the President months ago, these projects must be of a kind to produce permanent value and so far as possible to return their cost. Above all, they must be adapted to immediate initiation and to the efficient use of relief-roll labor, both skilled and unskilled, in large volume. It goes without saying that a high degree of directing skill must be provided, and since the cost limitation may render it difficult to provide this skill as part of the relief-works costs it will be helpful if it can be supplied by the local authorities. Without such cooperation the difficulties in the way of carrying through the program may prove insurmountable; they are serious enough as it is.

But while carrying out this national obligation the public-works engineer should realize clearly the service-improvement needs of his own community. Knowing that he cannot expect much help from the federal program, and knowing that the community's services deteriorate rapidly, he must proceed actively to plan essential improvements and put them under construction with direct financing. Money is available in ample amount, but not all citizens and public officials are yet convinced that it is the part of wisdom to put it to work on needed local improvements. To bring about this conviction is perhaps the most difficult part of the obligation now placed on public-works engineers, but it is also the most important.

A Bigger Crowbar

PRYING idle funds loose from bank vaults for home repair and modernization has been one of the principal tasks of the Federal Housing Administration since its creation a year ago. The crowbar used was in the form of insurance to lending institutions, to cover possible losses from loans up to $2,000 each. Recently amendments to the National Housing Act increased the limit to $50,000. This larger crowbar, it is hoped, will help move funds into industry, and to this end it is applicable to plant machinery as well as to building replacement and repair.

The toll of obsolescence and depreciation has accumulated rapidly over the past few years. There is urgent need that it be checked. Industrialists, large and small, as well as building owners and operators, appreciate this; but, as was demonstrated with the smaller modernization loans, it may be worth while to call the advantages of modernizing to their attention. In this task there is an excellent business opportunity for engineers and architects that constitutes at the same time a valuable public service. It is well within the bounds of professional practice and ethics to discover buildings that need modernization or repair, to make tentative studies of the profitableness of a modernization investment and to approach the owner with the results.

The incentive given lending institutions by insurance is real, and if business confidence is great enough to bring out the borrowers, a significant increase in construction repair work may result. The $50,000 limit is low, yet it brings a new opportunity for usefulness directly to the door of the engineer.

CURRENT NEWS

Construction League Protests Works Program Regulations

CHIEF RESULTS of a special meeting of the assembly of the Construction League held June 17 at Washington were the passage of a strong resolution of protest against the government's present plan for carrying out the relief-works program and revision of the League's organization to enable it to render a broader industry service now that the codes have been voided. An important feature of the reorganization is the formation of a Trade Practice Council as a division of the League, to formulate standards of practice applicable to competitive relations, inter-industry problems and the like.

The protest against the works program was drafted after the League was addressed on this theme by Willard Chevalier, vice-president, McGraw-Hill Publishing Co. Col. Chevalier traced the growth of stimulated public works since 1931 and the later combination of relief employment with the effort to continue public works after normal financing had failed. He commended the constructive purposes of the 1935 relief-works plan outlined by the President in his opening message to the present Congress, but showed by analysis of the actual CWA type of program defined by the recently announced cost limits that these purposes will not be realized and that the campaign will not only lack constructive effect but threatens to check industrial activity and resumption of normal investment.

Code replacement problems

One of the main purposes of the meeting was the consideration of methods by which the gains realized under the codes could be preserved. In discussion voluntary codes found no support, but the responsibility of the League for maintaining unity of the industry and developing rules for its guidance was definitely acknowledged. State integration under the leadership of state construction leagues was recognized as an important help in this work.

As a result of discussion two provisions for strengthening this leadership were included in a reorganization plan for the League: (1) To continue the local inter-group coordinating committees set up in 53 cities under sponsorship of the Construction Code Authority, these committees were authorized to affiliate with the League; it is intended that they shall deal with local problems as referred to them by the League's executive committee. (2) On motion of A. P. Greensfelder, of St. Louis, a Construction Trade Practice Council was set up as an agency of the League, to formulate generally acceptable standard practices applying to relations between industry and owner on public or private work, between divisions of the construction industry, between fellow employers, and in the field of employment.

In the general discussion of the subject

(Continued on page 892)

Bill Continues Resources Board As Permanent Federal Agency

(Washington Correspondence)

Given a renewed lease on life by Presidential executive order under the relief works act, the National Resources Board would be perpetuated as the National Planning Board in a bill proposed by Senator Copeland and endorsed by Secretary Ickes at a hearing before the Senate commerce committee, June 13.

"From my own experience," said Secretary Ickes, "I have appreciated keenly the value of such planning as has been done and the importance of broader and deeper planning for the future."

Established by statutory authority on a permanent basis, the board would function independently of any other government agency and would be composed of five members to be appointed by the President, to advise him with respect to plans and policies for the conservation of natural resources, and to cooperate with other federal, state and local agencies in such work.

Wagner Housing Bill Hearings Held Before Senate Committee

Both support and opposition to Senator Wagner's bill to create a permanent federal housing division in the Department of the Interior were voiced at the hearings on June 4, 5 and 6 before the Senate education and labor committee. The bill would authorize the federal government to aid low-cost housing through local housing authorities. An initial appropriation of $800,000,000 is provided for. Principal opposition came from Secretary Ickes in whose department the housing division would function. While approving the plan to place housing on a permanent basis, Secretary Ickes opposed the bill on the ground that it would not permit the division to initiate projects in localities where local housing bodies existed and that it deprived the housing division of control over projects financed by it through loans and grants in such localities. He also thinks loans to limited-dividend companies should not be prohibited.

Ickes Expects Substantial Non-Federal Program From Relief-Works Money

(Washington Correspondence)

IN ORDER to demonstrate that non-federal projects can qualify under the President's requirement that the cost shall not exceed an average of $1100 per man-year, Secretary Ickes announced this week that on the basis of the 45 per cent grant now authorized, the average cost of keeping one man at work one year *on the construction site* of the average type of PWA project would be $959. This was established by PWA following a study of 4,081 completed projects, embracing 72 different types of construction, based on actual expenditures and direct employment on the site. Cities which are unable to borrow the balance of 55 per cent over the grant, can be taken care of, he said, by loans from the revolving fund maintained by the sale of bonds purchased by PWA in financing previous projects. On this basis, Secretary Ickes expects that a substantial program of municipal public works will be carried out.

The cost per man-year on some typical classes of projects, according to the PWA survey, was as follows:

Streets	$2,534
Sidewalks and curbs	2,575
Sewage-disposal plants	2,684
Water mains	2,679
Municipal buildings	3,268
Hospitals	2,628
Housing projects	2,409
Grade-crossing elimination	2,729
Flood control	2,176

The slum clearance housing program, he said, will be exempted from the $1100 rule, and he expects that the allocation of $249,000,000, which is awaiting the President's approval, will be exhausted.

Commenting on the use of machinery, Secretary Ickes asserted that the substitution of manual labor was like turning off electricity to give employment to candle-makers, and as a long-range policy could not be justified. However, it may have temporary justification when there is an excess of man-power. Comparison of two PWA projects of similar type reveals that one on which manual labor was used employed 59 men for 90 days and cost 18 per cent more than the other on which machinery was used and where 21 men were employed for 30 days.

A new book by Secretary Ickes, titled "Back to Work, the Story of PWA," will be published June 25. In it he describes what has been accomplished with the original appropriation of $3,300,000,000. His purpose in writing, he says, was to inform many people that public works is something more than simply starting to build something, somewhere, somehow.

The relief-works program

At the meeting on Monday of the state administrators for the relief-works program, Harry L. Hopkins directed that periodical reports concerning projects contemplated and approved and the employment involved be submitted to Washington, to serve as a basis for directing the nationwide operations. For statistical purposes, the country will be divided into ten areas where complete records will be maintained by projects, concerning employment, man-hours, wages, commitments and other data.

Grand Coulee increase

Allotments recommended by the Works Program Advisory Committee on Monday

totaling $42,610,557 include $23,000,000 for Grand Coulee Dam.

The application of the Geological Survey for allotment of $7,828,000 from relief-works funds, to cover the first year increment of the 10-year mapping program adopted by the Board of Surveys and Maps has been returned for downward revision. Difficulties involved are the high proportion of non-labor cost and the ban on new equipment that will confine the program to equipment available. The Coast and Geodetic Survey's application for $8,678,253 still is hanging fire. This would include the geodetic control phase of the Board's program plus coast surveying.

PWA loan rescinded

PWA rescinded a loan of $1,600,000 to the Hueneme Dock Corp. of California for building harbor and terminal facilities on the Pacific Coast on the basis of a report by the Corps of Engineers holding that the project does not have "reasonable business promise of economic success." The law requires that loans on non-federal projects shall be reasonably secure, and the conclusion reached was that the corporation's profits from warehousing and stevedoring would be considerably less than estimates, making net income from all sources inadequate to meet interest and amortization charges.

Wisconsin Works Plan Defeated in Legislature

The efforts of Governor LaFollette of Wisconsin to provide a $209,000,000 state public works fund to be administered by a state finance authority proved unsuccessful when the state senate on June 13 defeated his bill by a vote of 19 to 14. The proposed plan, whereby the federal government was to supply $100,000,000 and the Wisconsin Finance Authority the balance through the issue of non-interest-bearing scrip backed by new taxes and local government bonds, was described in *ENR*, May 30, p. 790. The defeat of the LaFollette bill puts Wisconsin on the same basis as other states so far as receiving help from the $4,800,000,000 federal fund is concerned. Its share is estimated to be $88,000,000

The measure, which was designed to meet the present emergency, and which included also a long-term program for state planning, had received the approval of President Roosevelt. One feature of the program was the coordniated administration by which the works program could be increased, decreased or discontinued as the sag in the business curve might require.

Among the arguments against the bill brought out in the hearings was the doubtful constitutionality of the measure, especially in respect to delegating bonding power, and the burdensome effect of increased taxes at the present time.

Registration Law for Architects Passed in Maryland

A bill creating the Maryland State Board of Examiners and Registration of Architects, passed at the recent session of the Maryland General Assembly, has been signed by Governor Nice. Non-residents acting as consulting architects are exempt if they are legally qualified in their own state and if they act through a properly licensed architect in Maryland.

Table Rock Project Denied Extended Construction Time

A petition by the Empire District Electric Co. for a two-year extension of time to begin construction of the Table Rock hydro-electric project on the White River in Missouri was denied by the Federal Power Commission on June 11.

At the same time the commission issued an order requiring the company to show cause by July 15, 1935, why its license for the Table Rock project should not be terminated for failure to begin construction on December 1, 1934, as provided for in the license. The Federal Water Power Act provides that if a licensee does not begin construction of the project for which the license is issued within the time specified, the Power Commission must, after due notice, terminate the license. In denying the petition, the commission emphasized that the Empire District Electric Co. held priority on the site at Table Rock for twelve years under a preliminary permit and license. The two-year extension was denied primarily on the ground that the public interest was involved, and also on the commission's interpretation of the Water Power Act to bar "squatting" on power sites by private interests under permit or license when good-faith, intention and financial ability to construct a project were not shown.

The commission held that the company offered no evidence at a previous hearing in March of its good-faith, intention or its financial ability to construct the project, but rather undertook to rely on an alleged "implied agreement" when the license was issued in 1932 that an extension would be granted. It was also held that the company had not made the proper showing required by law in support of the proposed extension because the company would not commit itself to a forecast that actual construction would be begun in 1936.

WHERE THE MIDTOWN TUNNEL WILL REACH DAYLIGHT IN NEW YORK

Bids opened last week indicate that work will soon start on the New York approach to the Midtown vehicular tunnel now being driven under the Hudson River. Work will center in the large cleared area shown in this view, the site of the depressed approach plaza where the New York end of the tunnel will emerge into daylight. The plaza is bounded on the west (far) end by Tenth Ave., on the north (right) side by 39th St., and on the south by 38th St. Parts of cleared areas for the approach ramps leading from 42nd St. on the north and 34th St. on the south can be seen. Direct connections to the approach ramps will be made at 34th, 35th, 36th, 40th, 41st and 42nd Sts. Bridges will carry 37th, 38th and 39th Sts. over the ramps. The tunnel takes off from the depressed plaza at the corner of Tenth Ave. and 39th St. (upper right) running west under 39th St.

The bids which were opened June 12 by the Port of New York Authority, cover the construction of the approach ramps, the cross-street bridges, the depressed plaza, a 565-ft. section of cut-and-cover steel bent structure and 450 ft. of rock tunnel under 39th St., which will take the tunnel to the east end of the river section, just west of Eleventh Ave., now being driven by Mason & Hanger Co., Inc. A ventilation shaft at this point will be included in the new contract. Alternates were asked on cast iron or welded structural steel lining for the rock section (circular). Ten firms participated in the bidding, with the Cornell Contracting Corp., New York, submitting low bids of $2,770,204 for cast iron lining and $2,720,304 for structural steel. Other bids, ranging to a high of $3,851,534, were submitted (from second low to high) by Rodgers & Hagerty, Inc., Poirer & McLane Corp., Underpinning & Foundation Co., Inc., Del Balso Constr. Corp., Mason & Hanger Co., Inc., Peter F. Connolly Co., Rosoff Tunnel Corp., and the Carleton Co.

The alternate structural steel segmental lining for the rock tunnel section is a new development, consisting of ten segments and a key per ring 60 in. wide. Each segment is 5 ft. wide and about 9½ ft. long on the outside arc. A ½-in. skin plate and ⅜x5½-in. side and end plates make up the segment, stiffened by two radial ribs of half a 16-in. 64-lb. rolled section, flanges outstanding. The whole is shop fabricated by continuous welding. Bolts would be used for erection purposes only, and all circumferential and segmental joints would be continuously welded in the field.

Bill To Maintain Code Standards Brought To Senate

(Washington Correspondence)

SWEEPING authority to fix minimum wage rates and maximum hours of employment under all Government contracts for materials or construction is vested in President Roosevelt by the Walsh bill introduced this week in line with the administration's policy to maintain code standards so far as possible. The bill provides for inclusion in all contracts of a representation or agreement that the wage paid will not be less than the minimum and hours not more than the maximum designated specifically or by reference in the invitation to bid, and that no person under 16 years old will be employed.

The bill applies to all contracts or agreements made, extended or modified hereafter and prior to June 30, 1937, and includes, in addition to federal contracts, all contracts entered into by states, cities and other political subdivisions which obtain grants or loans of federal funds. Breach of wage and hour requirements will render the contracting party liable to the government for liquidated damages equal to the minimum wage. Violators also would be penalized by cancellation of the contract.

The National Association of Manufacturers contends that the Walsh bill contains practically the same defects as NIRA's code-making provisions which were invalidated in the Schechter decision since there is no standard to guide the President in determining the wages and hours to be imposed. Since the bill is written in general terms, it supersedes all previous legislation inconsistent with its terms and thus may operate to set aside various laws relating to hours of labor on public works, payment of prevailing rates as required by the Bacon-Davis Act and provisions of the $4,000,000,000 relief works bill.

The Treasury Department's procurement division has notified all government agencies that until this legislation is passed all bids must be requested on a basis that the contractor will agree to abide by regardless of wage and hour requirements established later. The National Association of Manufacturers takes the position that any one desiring to bid on government work is entitled to have his bid considered without agreeing to the conditions contained in the Walsh bill and that the bid of the lowest responsible bidder cannot be rejected on this ground only. Comptroller General McCarl is, of course, the appropriate agency for determining whether a government agency has improperly rejected a bid.

Engineers Registration Law Enacted in Connecticut

A bill establishing registration for professional engineers and land surveyors in Connecticut was signed, June 13, by Governor Cross. Connecticut is the 33d state to enact an engineers' registration law. Registration laws in ten additional states are being sponsored by the National Society of Professional Engineers.

SKYRIDE TOWER RAZED BY DYNAMITING

One of the 628-ft. Sky Ride towers at the Century of Progress, Chicago, was razed June 8 by dynamiting the two main cable connections to the concrete anchor. Parting of these cables permitted the flexing of the rigid steel cable network between the two towers which pulled the unanchored tower inward toward its twin 1,850 ft. away. In falling, the tower parted at the 200 ft. level where the cars were dispatched from tower to tower. The 3,100 tons of steel in the demolished structure is being sold for scrap. The second tower will be taken down soon.

Passengers riding the rocket cars during the exposition numbered 4,320,000 but this was not enough to pay out the $1,200,000 cost. The loss each year was unofficially reported at $100,000. The venture was participated in by five companies: Great Lakes Dredge & Dock Co., Mississippi Valley Structural Steel Co., Inland Steel Co., Otis Elevator Co. and John A. Roebling Sons Co. Robinson & Steinman, New York, were the designers and Joshua D'Esposito, Chicago, the supervising engineer. Demolition of all exposition structures, the towers among them, is being done by the Barker-Goldman-Lubin Co., Chicago.

Third Flood Control Dam Started On Winooski River in Vermont

Ground has been broken for the Waterbury dam, the third on the comprehensive flood control project of the Winooski River in northern Vermont. Following the disastrous flood of late 1927, the Army Engineers made a report advocating the building of seven flood-detention reservoirs on various branches of the Winooski. The first two dams, at Wrightsville and East Barre, are now nearing completion by war veteran units of the CCC under the direction of the First New York District Engineer. The whole project and the construction of these two dams were described in *ENR*, November 9, 1933, p. 551 and Aug. 30, 1934, p. 266. The Waterbury dam will be built by the same CCC and Engineer Corps organizations. It will be one of the largest earth dams in the East, having a maximum height of 150 ft. and containing about 2,000,000 cu.yd. of rolled fill.

Loup River District Requests Bids on Columbus Power House

Bids on the power house of the Loup River Public Power District at Columbus, Neb., will be received on July 3 at the district office in Columbus. The work, which will be let on a unit price basis, includes 19,000 cu.yd. of concrete, 25,000 sq.ft. of forms, 830 tons of reinforcing steel, 170 tons of structural steel and 35,000 cu.yd. of backfilling. Contractors may bid on any schedule or any combination of schedules.

Construction League Meeting

(Continued from page 890)

of replacing code action, Lewis H. Brown, chairman, Johns-Manville Co., asserted that there is a distinct place for a vitalized Construction League as an agency to develop a pattern for local action. The controlling needs of today, Mr. Brown stated, are to assure the flow of investment funds for construction by stimulating investor confidence, to work for reconstruction of the mortgage banking system on a modernized basis, to work toward better construction at lower cost, and to organize for industry action with respect to the government by utilizing voting strength as do such groups as the American Legion and the Federation of Labor.

Organization strengthened

On recommendation of a special committee headed by Malcolm Pirnie the form of organization of the League was amended in respect to scope of membership and voting strength. In addition to association memberships, a class of sustaining member was established, to which individual companies are eligible.

The membership will hereafter be grouped in six divisions: professional, general contracting, special contracting, production, distribution, and industry associations. Equal voting strength is assigned to these divisions. Within the divisions the allowance of votes and amount of League dues is to be fixed on the basis of association income. Sustaining members also have votes.

Other subjects discussed

Among other action by the meeting was endorsement of a resolution presented by the A.G.C. condemning the practice of charging contractors for bidding plans and specifications. Announcement was made by John Dickinson, assistant secretary of commerce, that the Bureau of Foreign and Domestic Commerce will set up, beginning July 1, an industrial service unit for construction; it is expected to deal with statistics and related industry data.

Formation of a committee on housing was authorized after a strong address by A. P. Greensfelder, who referred to the ineffectiveness of the present federal housing program and urged that the League should not merely oppose bad legislation but should advance its own concrete recommendations with respect to a housing policy.

Announcement was made by S. F. Voorhees, chairman of the Construction Code Authority, that the Authority has taken the first step to dissolve is corporate existence. Similar action has been taken by the divisional code authority for general contracting, which administered Chapter II of the construction code.

Washington Highlights

By Paul Wooton

With the nation's greatest coal strike in the balance the Guffey Bill becomes the most important measure before Congress. Chances of the adoption of the bill in a form and within a time limit that will satisfy the United Mine Workers are remote. Never before have the United Mine Workers been in a position to close such a large percentage of the country's coal mines, and thereby cripple industry and transportation to so great a degree.

Discovery that there is ample law to extend insurance of deposits makes efforts to delay the Banking Bill a little more difficult. Under a compromise, open market operations probably will be handled by the eight members of the Federal Reserve Board and five governors of regional banks. One of the claims being made for the banking bill is that it will make good real estate paper rediscountable thereby stimulating construction.

Stabilization of foreign exchange is not likely to be attempted, authorities believe, until more progress will have been made in removing barriers to the flow of international trade.

As approved by the House Committee the new Shipping bill carries operating and constructing subsidies to replace the present ocean mail subsidy. Ships built in American yards by the proposed U. S. Maritime Authority would be sold to operators for the amount the ships would cost were they constructed in foreign yards.

A $291,000,000 Rivers and Harbors bill has been reported by the Senate Commerce Committee. It carries 253 improvement projects.

Washington, June 18, 1935.

Connecticut Authorizes Bonds For Merritt Highway Work

A bill authorizing Fairfield County to issue bonds totaling $15,000,000 in order that it may obtain additional funds from the federal government for the completion of Merritt Parkway has recently been signed by Gov. Cross. By issuing the bonds the county will be entitled to receive $14,000,000 from the federal government and in this way it is expected to reduce from 15 years to 2 or 3 years the time necessary for completion of the parkway.

The parkway, which is to be barred to heavy trucks, will connect with the Hutchinson River Parkway in New York. According to the terms of the bill the highway commissioner, when the funds are provided, is to proceed immediately with construction and completion of Merritt Parkway as a through road for non-commercial vehicles from the New York state line across the county of Fairfield to the Housatonic River. It is intended that ultimately the parkway will run to Massachusetts and to Rhode Island.

The recently signed bill grants the county authority to sell bonds to the Reconstruction Finance Corporation, and also provides that although the state does not issue the bonds, it will eventually pay for them out of the funds the state intended to use in the next 15 years in building the parkway.

Southwest Texas Streams, Swelled by Heavy Rains, Inundate Large Areas

AN AREA of over 150 square miles in the state of Texas, covering roughly the territory west and south of the Colorado River and extending to the Rio Grande, was subjected to damaging floods last week. Highways and railways apparently suffered large losses. Subject to later check, the state highway department places the bridge damage at $1,500,000 to $2,000,000. The principal streams involved include the Colorado, Nueces, Frio, Llano, San Antonio and Guadalupe rivers. At Uvalde on the Nueces, some 80 miles west and south of San Antonio, the Missouri Pacific and the Southern Pacific railroads lost bridges; five of the six spans of the S. P. crossing are reported to have been washed out. The highway bridge at Crystal City, 35 miles south of Uvalde, was also badly damaged. Newspapers report a 40-ft. rise in the river at Uvalde.

The Colorado River at Austin rose 20 ft. in 2½ hours to a stage of 41 ft. Debris and houses passed over the spillway of the municipal dam. The bridge on the Fort Worth-San Antonio highway went out. Upstream, near Marble Falls, part of the power house at the Bachman Dam washed away. Highway bridges at Llano, Marble Falls and Montopolis went out. The Llano River, which joins the Colorado above Marble Falls, was reported at its highest stage since 1888; at Mason it washed out the relatively new bridge built at a cost of $115,000. The bridge at Casteel also went out. The crest of the Colorado River flood was at Columbus, 100 miles from the Gulf, on June 18. Because of its record height, further bridge damage appeared inevitable.

At Three Rivers, where the Frio joins the Nueces, the highest stage in many years occurred. All highway and rail traffic between San Antonio and Corpus Christi was suspended. At Kerrville, 50 miles northwest of San Antonio, five piers of a highway bridge under construction over the Guadalupe River were lost. The Rio Grande was at high stage but had caused little damage, according to reports, although the Pan American highway south of Laredo had to be closed for several days.

Total property damages, exclusive of highways and bridges, and covering industrial plants and other buildings in the flooded areas, were estimated at $2,000,000 at the time of going to press.

Ohio River Locks and Dams Get $9,500,000 Allotment

A sum of more than $9,500,000 for locks and dams in the Ohio River Division has been allotted from the $4,800,000,000 emergency relief fund appropriated by Congress.

This amount is in addition to the $5,000,000 recently allotted for construction of locks and dams in the Beaver River in Pennsylvania, which are to be a part of the proposed river-to-lake canal over what is known as Proposed Route No. 1.

The new funds will be distributed among the various projects as follows: $1,200,000 for the construction of the Emsworth Dam below Pittsburgh, Pa.; $2,800,000 for construction of Dams 8 and 9 in the Allegheny River near Pittsburgh; $2,000,000 for the Tygart River Dam near Grafton, W. Va., and $5,508,000 for the Gallipolis Dam in the Ohio River near the mouth of the Kanawha River, and the Winfield Dam in the Kanawha near Winfield, W. Va.

FIRST UNIT IN GERMANY'S ROAD PROGRAM OPENED

The new motor highway between Darmstadt and Frankfort-on-Main was recently dedicated as the first step in Germany's $750,000,000 work plan. Frankfort-on-Main is to be the center of a road system that is planned eventually to cover 5,000 mi. To date 219 mi. have been completed of the 937 mi. which are now under construction; 937 mi. more have been authorized and plans are being prepared for an additional 1,250 mi. The program is expected to take six years for completion.

The highways will have two 25-ft. paved roadways separated by a 16-ft. island. Each roadway is divided by a painted center stripe providing two lanes in each direction. All grade crossings with ordinary roads as well as with railways will be eliminated.

It is reported that over 100,000 men are employed on this road construction at present. These workers are not given a normal wage but are housed and fed in the same manner as those in the labor service.

William J. Wilgus Honored For Work as Relief Head

William J. Wilgus, who served as head of the works division of the Emergency Relief Bureau of the city of New York, from September, 1934, until May of this year, was honored for that service at a dinner held at the Engineers Club, New York City, on June 13. About 100 engineers were present.

Colonel Wilgus, at the urgent request of the mayor and the heads of the Emergency Relief Bureau, came out of retirement last year to take over the difficult task of reorganizing and directing the works division, a department of the relief administration, handling 100,000 men and spending $10,000,000 a month on relief work. An account of this great undertaking was published in our issue of May 30, p. 778.

For eight months Colonel Wilgus devoted all his energies to reorganizing and building up the personnel of the division, only to have it demoralized early this year by an aldermanic investigation of the handling of relief in New York City, an investigation that now is generally conceded to have been for muckraking purposes, rather than for fact-finding. During the investigation, in protest against the scandalous manner in which his staff was being treated, Colonel Wilgus resigned, but at the request of the heads of the bureau continued as director of the works division until after the investigation had been completed.

Arthur S. Tuttle, president, American Society of Civil Engineers, acted as toastmaster at the dinner. Varying phases of Colonel Wilgus' wide career as an engineer were reviewed by L. B. Stillwell and George W. Burpee, consulting engineers, New York, and by Willard Chevalier, vice-president, McGraw-Hill Publishing Co. Colonel Wilgus' work with the relief bureau was reviewed by Ole Singstad, a member of the Emergency Relief Bureau, Arthur V. Sheridan, chief engineer of the Borough of the Bronx, and J. P. H. Perry, vice-president of the Turner Construction Co. and until a few days ago deputy director of the works division. Mr. Perry commented especially upon the splendid morale that had been built up in the works division under Colonel Wilgus' direction.

In responding, Colonel Wilgus made no reference to his unhappy experience in the works division but expressed himself as being satisfied for having been able to render a public service to the city of New York in an emergency of this kind. He laid special stress on the need for engineering direction of the emergency relief operations of the entire country.

New Engineering Courses at Washington University

Special training for engineering administration, combining courses in the school of engineering and the school of business and public administration, will be offered at Washington University, St. Louis, Mo., beginning next year. Students will take a four-year course leading to a B.S. degree. The new course will include about 70 per cent engineering and 30 per cent business. The first year will be identical with that of other engineering courses.

SOCIETY CALENDAR

AMERICAN SOCIETY FOR TESTING MATERIALS, annual meeting, Detroit, June 24-28.

AMERICAN SOCIETY OF CIVIL ENGINEERS, annual convention, Los Angeles, July 3-5.

AMERICAN SHORE AND BEACH PRESERVATION ASSOCIATION, summer meeting, Ocean City, N. J., July 11-13.

INSTITUTE OF TRAFFIC ENGINEERS, sixth annual meeting at Louisville, Ky., October 15-16.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, annual convention, Miami, Fla., December 9-12.

PENNSYLVANIA SEWAGE WORKS ASSOCIATION, annual meeting, State College, Pa., June 24-26.

NEW ENGLAND WATER WORKS ASSOCIATION, fall convention, Providence, R. I., Sept. 17-20.

THE WESTCHESTER (N. Y.) ARCHITECTURAL AND ENGINEERING ALLIANCE has elected FRANCIS R. MACLEAY, president, CHARLES DEWEY and JOSEPH L. FITZPATRICK, vice-presidents, MARY B. REVERE, corresponding secretary, WILLIAM C. STUART, recording secretary, and FRANK J. HEFFERNAN, treasurer.

THE ENGINEERS CLUB OF MONTGOMERY, Ala., was organized on June 5, with members from the city and from Montgomery County. HERBERT SPENCER, of the state highway department, was made president, A. L. FREEDMAN, O. L. SKINNER and IRA GREENHILL were made vice-presidents and REESE HARVEY, secretary-treasurer.

THE CLEVELAND ENGINEERING SOCIETY has elected JOSEPH H. GEPPERT, president, FRED L. PLUMMER, vice-president, and ARTHUR H. VAN HORN, treasurer.

CANTON, OHIO, CHAPTER of Professional Engineers and Land Surveyors at a recent meeting elected FRED L. BIECHELE, president, KARL SNYDER and LUTHER K. ZERBE, vice-presidents, ROY B. CLARKE, treasurer, and HOMER J. ANDERSON, secretary.

THE ENGINEERS CLUB of Birmingham recently elected the following officers: L. M. SMITH, president; G. R. OZLEY, vice-president; J. W. COTLIN, secretary-treasurer.

Brief News

THE AMERICAN INSTITUTE OF STEEL CONSTRUCTION has reestablished three of its district offices which were closed during the depression years. These new offices are at Dallas, St. Louis and Atlanta. L. H. Dodd will be in charge at Dallas. The personnel at the other two offices is undetermined as yet.

A $2,000,000 HIGHWAY, 17 MILES LONG, will be started in Massachusetts this summer to connect the Worcester-Millbury city line with the Rhode Island highway system south of Uxbridge. A four-lane highway will be built with a grass plot in the center, as on the recently completed Boston-Worcester route. The present traffic is carried on a two-lane highway through the congested Blackstone Valley. The new road will, for twelve miles, be by-passed around town centers.

THE UPPER COLORADO RIVER AUTHORITY has been created by a bill signed by Gov. James V. Allred of Texas. State taxes of approximately $65,000 annually in Tom Green and Coke counties will be remitted for 20 years as a guaranty for Federal funds for construction of dams and reservoirs, for irrigation and water and soil conservation. The Authority hopes to get $4,000,000 from the Federal government. The tax remission will be revoked if the district does not obtain $2,000,000 Federal funds by 1937.

Personals

ROSS WHITE, construction engineer on Norris Dam, has been appointed as general superintendent of construction of all TVA projects.

ERNEST H. HARVEY, former designing engineer for the city of Newton, Mass., has been appointed as city engineer. He succeeds WILLIAM P. MORSE wha has retired.

LIEUT. COL. S. C. GODFREY of the Engineer Corps at Boston went to Washington on June 1 for temporary duty in the office of the Chief of Engineers and will go to the Panama Canal Dept. in July.

HARRY S. BERRY, former state highway commissioner of Tennessee, has recently been made Works Progress Administrator for that state. From August to December, 1933, he was PWA engineer for Tennessee. He then went to Washington as an assistant deputy NRA administrator and continued in that position until the present change.

CHARLES R. GOW, president Warren Bros. Co., Cambridge, Mass., and active for many years in engineering construction and public affairs in Boston, was awarded an honorary degree of doctor of engineering by Worcester Polytechnic Institute.

W. A. KELLY, who for several years has been engineer appraiser for the Federal Land Bank of St. Louis, Mo., has been appointed senior hydraulic engineer for the Land Bank Division of the Farm Credit Administration, Washington, D. C. Previously Mr. Kelly was engineer appraiser for the Federal Land Bank of Omaha. Before his work with the Omaha bank, Mr. Kelly was for seven years with the Office of Drainage Investigations of the Bureau of Public Roads.

EDWARD E. WALL, consulting engineer and director of public utilities, St. Louis, Mo., was granted an honorary degree of doctor of laws from the University of Missouri from which he was graduated in civil engineering in 1884.

VICTOR L. STREETER, junior engineer of the U. S. Reclamation Bureau of Denver, Colo., has been awarded the John R. Freeman Scholarship which is offered annually by the American Society of Mechanical Engineers. The scholarship provides for fifteen months' travel in Europe with research work in hydraulics. Mr. Streeter, who received his doctor of science degree from the University of Michigan in 1934, has been with the Reclamation Bureau since that time. The scholarship was founded by the late John R. Freeman, who was widely known for his work in hydraulics.

JAMES M. S. WARING has been appointed economic adviser of the executive staff of the Division of Applications and Information of the Relief Works Administration. Mr. Waring, who received his civil engineering degree from Virginia Military Institute was, from 1910 to 1927, a consulting engineer with L. I. Summers Co., New York and Chicago. In 1933 Mr. Waring was a member of the regional

advisory board of the PWA and in 1934 he was with the division of research, statistics, and information of the FERA.

JOHN E. OSTRANDER, head of the department of mathematics at Massachusetts State College, retired on June 10 after 38 years at the college. He had reached the retirement age of 70 set for state employees. At the commencement exercises he was appointed Emeritus Professor of Mathematics. Professor Ostrander, a graduate of Union college in 1886 with the degree of C.E., received his A.M. in 1889. After several years of private practice as a civil engineer he became professor of civil engineering and mechanic art at the University of Idaho in 1892. He went to Massachusetts State College in 1897 where he served as meteorologist.

Obituary

HINES M. SMITH, who had participated in the building of the Southern R.R. from Chattanooga to Atlanta and who was widely known in the south for bridge and railway construction, died June 9 in Chattanooga, at the age of 84.

GEORGE MERRICK, sanitary engineer of Salem, Mass., died in that city on June 11 at the age of 62. He was born in England and had been connected with the John F. Cabeen Co., in Salem, for many years while engaged in engineering work.

CHARLES P. DICKINSON, promoter and builder of four hydro-electric plants on the Rondout Creek in New York, and for about a quarter-century sole owner of the Honk Falls Power Company, which served Kingston, Newburgh and Poughkeepsie with electricity until its sale to the Central Hudson Gas & Electric Company in 1920, died in Boston June 11 at the age of 81.

ALAN MCDONALD, a member of the San Francisco firm of McDonald and Kahn, died suddenly June 8 at Los Angeles, Calif. He was 51 years of age. Mr. McDonald was one of the organizers of the Six Companies, Inc., builders of Boulder Dam. In San Francisco he was a consultant on the building of the Golden Gate and Bay bridges and was prominently identified with other Bay region construction activities. He was born in Louisville, Ky., in 1883 and was a graduate of Cornell University. He had been a resident of California since 1908 and has been actively engaged in construction on the West Coast.

CONSTRUCTION STATISTICS OF THE WEEK

ENGINEERING construction awards at $32,279,000 are higher this week due to higher public awards. Public totaled $26,193,000, of which $13,982,000 represents state and municipal and $12,211,000 federal. Private projects totaled $6,186,000, a 50 per cent decline from last week.

Public buildings at $9,223,000 and earthwork and waterways at $5,395,000 are considerably higher than last week. There has also been an increase in streets and roads at $4,327,000, bridges at $2,611,000, unclassified at $3,277,000 and waterworks at $693,000.

The larger awards include: electrical equipment for sheet steel strip mill at Ecorse, Mich., for the Great Lakes Steel Co. (two contracts) $2,500,000; federal housing project at Indianapolis, Ind., $2,241,000; at Cleveland, Ohio, $2,476,000; portable fabric dwellings to be built by Constructing Quartermaster of West Texas District for Citizens Conservation Corps in Texas, estimated to cost $637,000; highway awards by West Virginia, $624,000. At New York City the Triborough Bridge Authority let the contract for the superstructure of the Harlem River Bridge Crossing of the Triborough Bridge, $1,314,000; and at Colorado Springs, Colo., the city will rebuild five bridges washed out in the recent flood at an estimated cost of $500,000. Divisions J, K, L and M of the Calumet sewage-treatment works, Chicago, Ill., have been put under contract at $883,000. U. S. Engineers will construct the Little River Flood Control Dam at Waterbury, Vt., estimated to cost $3,600,000 by day labor; also let contract for relocating portion of Oregon, Washington R.R. and Navigation Co. between Mile Post 33 and Cascades Lock near Portland, Ore., $793,000; 565,000 bbl. portland cement for Bonneville Dam, $917,000. The Department of Water and Power, Los Angeles, Calif., let a contract for a frequency changer at $632,000.

New capital for the week totaled $7,898,000, of which $7,658,000 is state and municipal and $240,000 federal.

CONTRACTS
(Thousands of Dollars)

	Weekly Average June 1934	Prev. 4 Weeks	Week June 20 1935
Federal Government	$5,624	$4,812	$12,211
State and municipal	17,343	11,652	13,982
Total public	$22,967	$16,464	$26,193
Total private	4,531	11,361	6,186
Week's total	$27,498	$27,825	$32,379
Cumulative to date:			
1934...$680,919,000	1935...$625,446,000		

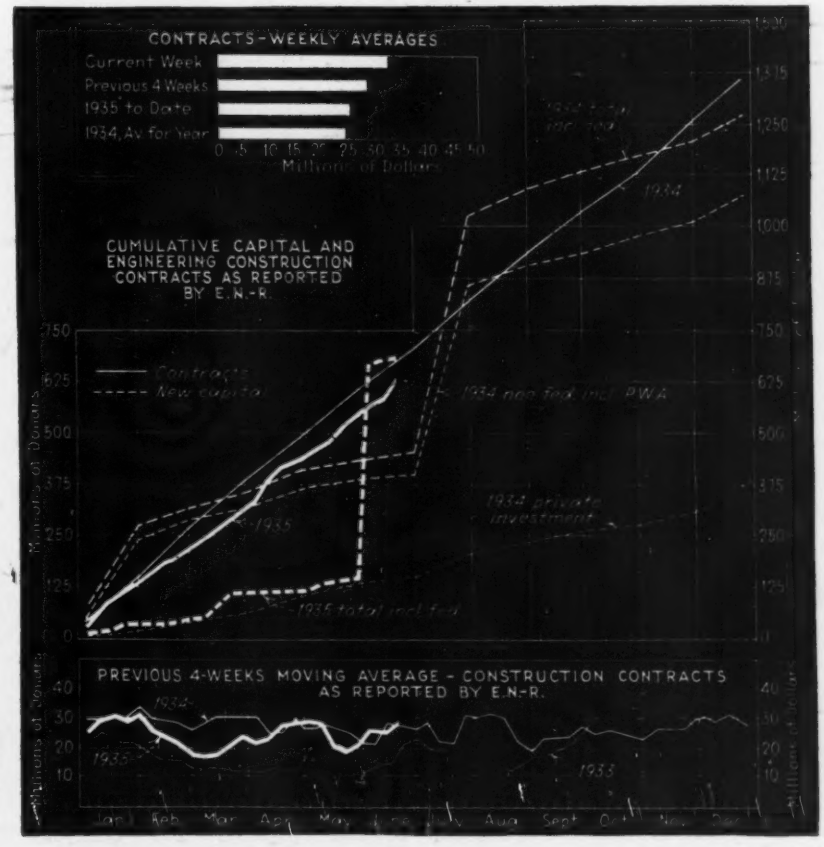
NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

1935	Week June 20	Cumulative
State and municipal	$7,643	$191,331
PWA allotments, S&M	15	—49,017
RFC loans, S&M	....	84
Corporate issues	....	....
PWA allotments, private	....	8,619
Total, Non-Federal	$7,658	$151,017
PWA allotments, Federal Constr.	240	532,088
Total new capital	$7,898	$683,105
Cumulative to date:		
1934...$452,104,000	1935...$683,105,000	

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for Federal construction. Negative amounts in PWA totals indicate excess of rescissions and reductions over reallotments and increases in outstanding allotments.

INDEX NUMBER

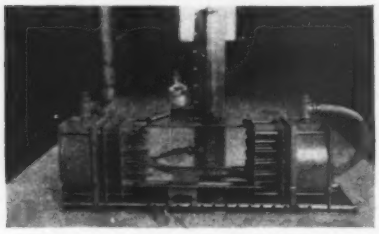
E.N.R.-Cost	1913 = 100	1926 = 100	E.N.R.-Volume	1913 = 100	1926 = 100
June, 1935	194.78	93.63	May, 1935	109	52
May, 1935	194.06	93.28	Apr., 1935	130	57
June, 1934	199.61	95.95	May, 1934	101	44
1934(Av.)	198.10	95.23	1934(Av.)	114	50
1933(Av.)	170.18	81.80	1933(Av.)	102	47
1932(Av.)	156.97	75.45	1932(Av.)	127	56


CONTRACTS - WEEKLY AVERAGES
Current Week
Previous 4 Weeks
1935 to Date
1934, Av for Year
Millions of Dollars
CUMULATIVE CAPITAL AND ENGINEERING CONSTRUCTION CONTRACTS AS REPORTED BY E.N.-R.
Contracts
New capital
1934 private investment
1935 total net for
PREVIOUS 4-WEEKS MOVING AVERAGE - CONSTRUCTION CONTRACTS AS REPORTED BY E.N.-R.
1934
1935
1933
Millions of Dollars
Jan Feb Mar Apr May June July Aug Sept Oct Nov Dec

Construction Equipment and Materials

Core Measuring Instrument Invented By N. Y. Chemist

The Corometer, a device for automatically measuring the average thickness of cores taken from finished pavements, has been developed by W. G. Doherty, a chemist in the Bronx Testing Laboratory. The sales agent for the Corometer is the T. H. Williams Co., Bronx, N. Y. The instrument consists of two barrel-like cylinders mounted on a rectangular base so that they can slide toward each other. Projecting from each barrel, parallel to the base, are twenty-five steel rods that move in and out easily. These rods are well distributed



COROMETER — for determining average thickness of cores.

within the area of a 6-in. circle in order to measure a core of approximately 6-in. diameter. The barrels contain a heavy oil. On top of the oil is alcohol which is guided through a tube from each barrel to a single gage glass calibrated in fractions of an inch.

To measure a core the two barrels are first brought to rest against a pin which has been placed in a hole marked zero in the base. By means of a leveling bottle the level in the gage is also brought to zero. The two barrels are now spread apart and the core placed on a slideable "V" block between the units. The barrels are forced toward the core until all rods touch points on the core, and the movement is then continued until the right unit is stopped by the limit pin which had been placed in the 3-in., 4-in. or 5-in., etc. hole in the base, depending on the thickness of the particular core. As the fingers come in contact with the ends of the core, they are forced into the barrels, thereby displacing liquid, which is translated into inches on the gage glass. This reading is added to the number marked on the base at which the limit pin was placed for the measurement. The sum of these two readings is the average thickness of the core under test.

This instrument checks the core measurement to within one or two hundredths of an inch regardless of the irregularity of the core. The device is self-calibrating, and the gage markings may be verified in a few minutes.

Concrete Vibrator Designed To Reach Into Deep Forms

A new type electric combination internal, mass and form concrete vibrator for placing low slump, mass and form concrete has been announced by the Mall Tool Co., Chicago, Ill.

The motor and vibrator in this unit are short-coupled, and the entire machine can be carried and operated by one or two men. The machine weighs 140 lb., and the overall length is 5 ft. 6 in. As a flexible shaft vibrator the same motor as used with the short-coupled vibrator can be utilized to drive small-sized vibrators through flexible shafting for vibrating concrete in the forms or in places difficult to reach with the larger mass concrete vibrator. The motor can be used with either 14 ft., 21 ft., or 28 ft. of flexible shafting, depending upon the requirements in reaching the vibrator into the form. The Mall concrete vibrator is designed to give faster, more economical placement of stiff concrete mixes and denser, stronger, and more durable concrete.

Backfill Scraper For Cutting Through Hard Material

A new type backfill scraper has recently been developed by the Newo Hoist Co., Cleveland, Ohio. This scraper has a heavier and more rigid construction than the conventional type. There are no side bars or braces to keep the cutting edge from entering the material freely, and because of this the manufacturer claims the scraper will dig into harder material than most backfillers. The absence of a chain bridle on the front keeps the scraper from catching on obstructions. The Newo scraper is built in widths of 3 ft. to 12 ft., to fit all makes of cranes.

New Equipment in Brief

Combined Electric Hammer and Drill. A new idea in portable electric tools is the Wodack "Do-All" combination electric hammer and drill made by the Wodack Electric Tool Corp., Chicago. The hammer mechanism has only two working parts, which are made of treated steel. By a simple operation the hammer may be removed and the tool converted into a drill with a capacity of ⅝ in. in metal. When used as a hammer with star drills the tool will make cuts up to 1⅝-in. diameter in concrete.

Road Shaper—A medium-weight road shaper, built with two blades at opposing angles, has been developed by the Gledhill Road Machinery Co., Galion, Ohio. The opposing blades eliminate side draft and the rear blade mixes the material and rolls it into ruts and low places. Straight-edges on each side of the machine keep the blades true and even, shaping the road to a true level. The four-wheeled chassis is designed to be drawn behind a 1- and 1½-ton truck.

Tractor Shovel. A small shovel mounted on a standard crawler tractor produced by the Speeder Machinery Corp., Cedar Rapids, Iowa, presents a new idea in excavation equipment. The "Caterpillar" Diesel 40 tractor furnishes the power for the shovel and enables it to negotiate soft ground and ditches. The shovel boom is 14 ft. and the dipper stick 11 ft. long. Speeder cranes and draglines, similarly mounted with 22-ft. booms, are also available.

Gravel Plant. Pioneer Gravel Equipment Mfg. Co., Minneapolis, Minn., announces the 1935 model Pioneer No. 15 duplex gravel crushing, screening and loading plant. The plant requires only one power unit. A shaft and gear drive is employed wherever feasible. The return conveyor system is employed instead of a bucket elevator. The primary crusher is of the jaw type, and a roll crusher operates as the secondary.

Business Notes

F. C. CRANE COMPANY, Dallas, Texas, has been appointed by the Bucyrus-Erie Co. for central and eastern Texas.

A. J. O'LEARY has been advanced to assistant general manager of sales for the Lukens Steel Co., Coatesville, Pa.

T. A. CANTY, Baltimore, Md. distributor for the Lincoln Electric Company of Cleveland has moved his office to 1023 Cathedral Street.

ALFRED RENNOLDS, for many years identified with construction machinery activities in the Middle-West, has taken over the State Line Machinery Co., which he will operate as the Rennolds Machinery Co. in Chicago.

LEE WRIGHT has been appointed sales representative for Republic Steel Co. with headquarters in Salt Lake City.

THE SHADBOLT & BOYD CO., Milwaukee, Wis., has received the appointment as agent in the Milwaukee region for the General Refractories Co., Philadelphia, Pa.

THE SERVICE SUPPLY CORP. and the Rental Service Co., Philadelphia, Pa. have opened a branch warehouse for contractors equipment in Harrisburg, Pa.

JOHN F. GOWEN assumed his duties as executive secretary of the Producers Council New York City on June 3. Mr. Gowen is associated with Lehman Brothers, investment bankers.

THE C. R. JAHN COMPANY, manufacturers of heavy duty equipment trailers has moved its offices from Chicago to the Funk Bldg., LaCrosse, Wis.

THE BROWN INSTRUMENT CO., Phila., Pa. and the MINNEAPOLIS-HONEYWELL REGULATOR CO., Minneapolis, Minn., have opened a joint sales office in Atlanta, Ga. Mr. Wesley R. Moore will be in charge of the Atlanta office.

THE J. G. WHITE ENGINEERING CORP. has moved its offices from 43 Exchange Place to 80 Broad Street, New York City.

EDWARD M. ADAMS, vice president in charge of sales for the Inland Steel Co., Chicago, died in Hot Springs, Ark., May 8.

New Publications

WORTHINGTON AIR COMPRESSOR UNITS, TYPE VS, Worthington Pump and Machinery Corp., Harrison, N. J. 8½x11, 4 pages.

NEWARK WIRE CLOTH FOLDER, Newark Wire Cloth Co., Newark, N. J. 8½x11, 4 pages. Contains details of standard sieves.

BEAUTY IN WALLS OF ARCHITECTURAL CONCRETE, Portland Cement Association, Chicago, Ill. 8½x11, 24 pages.

EFFICIENT REGULATION OF AN AIR-CONDITIONING SYSTEM, Leeds & Northrup Co., Philadelphia, Pa. 7½x10. 24 pages.

THE NICHOLS HERRESHOFF INCINERATOR FOR SEWAGE SLUDGE DISPOSAL, Nichols Engineering & Research Corp., New York. 8½ x11, 12 pages, Bulletin No. 203.

THE LIMITLESS FIELD FOR HAYWARD BUCKETS, The Hayward Co., New York, N. Y. 3½x6½. 24 pages. Bulletin 300.

DIEHL SIMPLIFIED PRICE LIST AND CATALOG, Diehl Manufacturing Co., Elizabethport, N. J. 8½x11, 18 pages.

CATALOG NO. 1542, Link-Belt Co., Chicago, Ill. 16 pages. Straightline Collectors for removing sludge and scum from tanks at sewage and water treatment plants.

SILVER ANNIVERSARY DELUXE CATALOG, Black & Decker Manufacturing Co., Towson, Md. 8½x11, 50 pages. Portable Electric Tools.

Current Construction Unit Prices

Oregon Coast Highway Bridges

CONSTRUCTION of the five remaining bridges required to complete the Oregon Coast Highway is under way, financing being met with a federal grant of 30 per cent and a self liquidating loan amortizing over a 28-year period as to principal and 4 per cent interest for the remainder of the total cost approximating $5,500,000. Estimates indicate that gross receipts will be sufficient to operate and maintain the briges, to meet all principal and interest payments when due and in addition to accumulate a reserve of $1,000,000 in a 12-year operating period. A brief description of each structure follows supplemented by tabulations of the unit prices and quantities upon which each contract was based.

Yaquina Bay Bridge—Consists of a series of three steel arch spans flanked on either side by reinforced concrete arch and deck girder construction. The channel span is 600 ft. between centers of end pins, and the flanking spans 350 ft. The balance of the structure, which is 3260 ft. overall, is of reinforced concrete arch and viaduct construction. The central arch's crown height will be 246 ft. above mean sea level. Contract for this bridge was awarded July 25, 1934, for $1,357,588 to the General Construction Co., Seattle, in combination with Gilpin Construction Co., Portland, Ore.

Alsea Bay Bridge—Consists of a series of three concrete arches of the tied or bow string type, flanked on either side by a group of three 150-ft. deck arch spans which are flanked in turn by reinforced concrete viaduct approaches. The total length overall is 3028 ft. and the central group of arch spans has a vertical clearance of 70 ft. for water craft and horizontal clearances of 200 ft. and 143 ft. for the central and flanking spans. Contract was awarded April 26, 1934, in the amount of $685,040 to a combination of Lindstrom & Feigenson, Parker & Banfield and T. H. Banfield, all of Portland, Ore.

Siuslaw River Bridge—Consists of a central electrically operated bascule having a clear span of 140 ft. flanked on either side by reinforced concrete arch construction of the bowstring type which in turn are flanked by reinforced concrete viaducts. Contract for this structure with an overall length of 1360 ft. was awarded July 25, 1934, to Mercer-Fraser Construction Co., Eureka, Cal., in the amount of $491,646.

Umpqua River Bridge—Consists of a steel swing draw bridge with an overall length of 430 ft. flanked on either side by two concrete bowstring spans of 154 ft. each. The balance of the structure which is 2213 ft. overall consists of reinforced concrete deck construction on creosoted timber pile trestles through which a 40-ft. opening is being left for the passage of small boats. Contract totalling $551,234 was awarded July 25, 1934, to Teufel & Carlson, Seattle.

Coos Bay Bridge—Consists of a central steel cantilever span having an overall length of 1708 ft. flanked on either side by reinforced concrete arch viaducts. The total length of the structure is to be 5337 ft. and contract was let July 25, 1934, in two sections, the first for the piers and approaches to the Northwest Roads Co., Portland, Ore., for $1,529,438 and the second for the steel cantilever span to the Virginia Bridge & Iron Co., Roanoke, Va., for $593,880.

Waterworks at Saratoga Springs, N. Y.

WATER SUPPLY improvements at Saratoga Springs, New York, proposed under contracts awarded early in 1935, include a pumping station and filtration plant to be situated immediately south of the D.&H. R.R. at Loughberry Lake, and a distribution reservoir at Woodlawn Park, approximately 1 mile northwest of the new waterworks.

The project is divided into three main subdivisions: buildings and structures, distribution reservoir, and transmission main. The substructure of the filter, head and pump house, including the chemical feed room is T-shaped, extends 22 ft. below grade, is 165 ft. in length overall, with widths of 39 and 63 ft. A second story, 50x57 ft., is located over the head house and chemical feed room. Superstructure construction consists of 12-in. brick bearing walls, 5-in. reinforced concrete floor slabs with 1½-in. finish, and steel truss roofs with slate and copper decks. Windows are of wood with cast-stone trim. The portico, 31 ft. 8 in. x 14 ft., has stone steps, cement finish, and columns of wood extending 21 ft. to wood cornices. Interior finish in the filter, head and pump houses consists of terazzo floors and base, plaster walls and cornice, vaulted plaster ceilings, wood trim and applied wood paneling. The chemical storage room has granolithic finish floors with exposed roof trusses at ceiling and painted roof soffits. The volume of the superstructure totals approximately 177,000 cu.ft. showing an indicated cu.ft. price of 36c. for this section of the building in terms of the low bid.

The distribution reservoir is 212x152 ft. with an average depth of 27 ft. Exterior and partition walls are of 18-in. reinforced concrete. The floor is 8 in. thick, and the roof is 6½ in. thick, both reinforced.

Work is scheduled for completion within 260 days after award of contract for the buildings and structures. The

CONTRACT PRICES—OREGON COAST HIGHWAY BRIDGES

Item	Yaquina Bay Bridge Quantity	Unit	Alsea Bay Bridge Quantity	Unit	Siuslaw River Bridge Quantity	Unit	Umpqua River Bridge Quantity	Unit	Coos Bay Bridge Quantity	Unit
1. Excavation—cu.yd	25,000	$8.75	7,000	$12.00	3,800	$17.00	3,500	$12.00	24,000	$8.00
2. Excavation, added depth—cu.yd	1,500	5.00	200	12.00	300	2.00	300	12.00	2,000	8.00
3. Foundation piling—lin.ft	100,000	.50	82,000	.46	34,500	.62	23,000	.575	190,000	.50
4. Tr. fender piling—lin.ft	21,000	.70	.....	.....	6,300(b)	.62	18,000(b)	.53	27,000	1.25
5. Concrete piling—lin.ft	2,000	3.25	.....	.....	.....	.....	.....	.....	.....	.....
6. Seal concrete—cu.yd	3,800	17.75	2,200	18.50	1,500	25.00	1,200	10.00	8,000	12.00
7. Concrete, Cl. A, piers, footings—cu.yd	11,000	17.75	.....	.....	.....	.....	.....	.....	19,000	15.80
8. Concrete, Cl. A, pier shafts—cu.yd	4,200	20.00	.....	.....	.....	.....	.....	.....	5,200	18.00
9. Concrete, Cl. A, beams, colums—cu.yd	4,300	29.50	.....	.....	4,500	28.000	3,400	25.00	8,000	27.80
10. Concrete, Cl. A, arch ribs, struts—cu.yd	2,400	31.25	4,400	29.00	800	38.00	1,600	28.00	6,400	29.00
11. Concrete, Cl. D, concrete spans—cu.yd	1,570	27.00	2,800	22.00	1,400	25.00	1,815	25.00	3,040	23.50
12. Concrete, Cl. D, steel spans—cu.yd	930	22.75	.....	.....	.....	.....	285	24.00	1,160	26.00
13. Concrete, Cl. B—cu.yd	.....	.....	6,600	17.70	1,600	21.00	1,600	14.00	.....	.....
14. Concrete, Cl. A, approaches—cu.yd	.....	.....	2,900	26.00	.....	.....	.....	.....	.....	.....
15. Concrete handrail—lin.ft	6,750	5.00	6,200	4.00	3,000	4.00	3,600	4.00	7,500	3.50
16. Metal reinforcing—lb	2,250,000	.045	2,000,000	.045	1,050,000	.042	1,300,000	.055	4,012,000 398,000(a)	.0465 .0435
17. Cast chromium steel—lb	20,000	.35	.....	.....	17,000	.26	30,000	.34	18,000 8,000(a)	.48 .4035
18. Struct. carbon steel, copper bearing—lb	2,830,000	.073	.....	.....	600,000	.066	1,060,000	.073	4,445,000	.0742
19. Struct. silicon steel, copper bearing—lb	1,300,000	.0765	.....	.....	.....	.....	420,000	.088	2,800,000	.0762
20. Structural steel	.....	.....	300,000	.08	.....	.....	.....	.....	20,000	.10
21. P. O. cedar lumber—M B M	.....	.....	.....	.....	52	90.00	90	70.00	.....	.....
22. Machinery—lb	.....	.....	.....	.....	85,000	.21	105,000	.27	.....	.....
23. Electrical and misc equip.—complete	.....	.....	.....	.....	Lump	16,000.00	Lump	21,364	.....	.....

(a) Steel for cantilever span. (b) P.O. cedar fenders.

following tabulations list the units, totals and quantities of the three low bids applying on each division of the project. The work is to be under the direct supervision of City Engineer Samuel J. Mott. Plans were prepared by Nicholas S. Hill, Jr., consulting engineer, New York.

SARATOGA SPRINGS WATER IMPROVEMENTS—LOW BIDDERS

I—Buildings and Structures	
A—F. H. McGraw & Co. Inc., New York, N. Y. (contract)	$171,075.46
B—Thos. C. Brown Co. Inc., Schenectady, N. Y.	171,466.32
C—Adams Bros. Const. Co., Glen Falls, N. Y.	179,958.00
II—Transmission Main—Section A	
D—Florence Pipe Foundry & Machine Co. (contract)	$42,579.99
E—U. S. Cast Iron Pipe & Foundry Co., Burlington, N. J.	44,021.04
F—Warren Foundry & Pipe Co., Phillipsburg, N. J.	45,739.70
III—Transmission Main—Section B	
G—Ludlow Valve Mfg. Co., Troy, N. Y. (contract)	$3,651.74
H—Iowa Valve Co., Des Moines, Ia.	3,563.01
J—Darling Valve Mfg. Co., Williamsport, Pa.	3,647.19
IV—Miscellaneous	
K—Chas. Miller & Son Co., Utica, N. Y. (contract)	$1,584.00
L—Aird Don Co., Troy, N. Y.	1,736.00
M—Interstate Plumbing Supply Co.	1,800.00
V—Distribution Reservoir	
N—Collins Bros., Mechanicville, N. Y. (contract)	$102,224.25
P—F. H. McGraw & Co. Inc., New York, N. Y.	102,442.50
R—Warren Bros. Roads Co., Boston, Mass.	114,185.45

UNIT PRICES—SARATOGA SPRINGS WATER IMPROVEMENTS

I—Buildings and Structures	A	B	C
1. Excavation & Backfill—19,000 cu. yd.	$0.35	$1.05	$0.75
2. Earth embankment—35,000 cu. yd.	0.35	0.3888	0.85
3. Sheeting left in place—15 M. B. M.	60.00	43.30	58.00
4. Seeding—1 acre	450.00	348.00	380.00
5. Roadways & paths—2,000 sq. yd.	1.00	0.95	1.75
6. Concrete masonry, Class A—3,100 cu. yd.	23.00	21.70	24.25
7. Concrete masonry, Class B—100 cu. yd.	15.00	14.25	14.00
8. Portland cement—100 bbl.	2.60	2.60	4.00
9. Steel reinforcement—10,000 lb.	0.04	0.0375	0.04
10. Brick masonry, manholes, etc.—40 cu. yd.	20.00	23.90	26.00
11. Furnish & lay outside piping, 2 in. C.I.—130 l.f.	0.53	1.33	0.57
12. Furnish 4 in. & 6 in. C. I. Class B—120 l.f.	1.19	1.69	1.30
13. Furnish 8 in. C. I. Class B—250 l.f.	1.68	2.56	1.65
14. Furnish 12 in. C. I. Class A—320 l.f.	2.81	4.10	2.85
15. Furnish 16 in. C. I. Class B—320 l.f.	4.21	7.47	4.25
16. Furnish 16 in. C. I. Class C—70 l.f.	4.84	6.30	4.85
17. Furnish 20 in. C. I. Class C—12 l.f.	6.68	9.66	7.00
18. Furnish 24 in. C. I. Class B—470 l.f.	7.72	10.71	7.70
19. Furnish 30 in. C. I. Class B—80 l.f.	10.60	16.68	10.50
20. Furnish & install outside valves in hydrant—lump sum	600.00	675.00	550.00
21. Furnish & lay outside piping, 6 in., vitrified—300 l.f.	0.28	0.48	0.35
22. Furnish 8 in. vitrified—1,100 l.f.	0.37	0.53	0.50
23. Furnish 10 in. vitrified 1050 l.f.	0.52	0.774	0.60
24. Furnish & lay 24 in. reinf. concrete pipe—200 l.f.	2.90	3.96	3.25
25. Manhole frames, covers, gratings, guides, etc.—lump sum	2,100.00	2,210.00	1,925.00
26. Furnish & place screens & hoist—lump sum	700.00	897.00	1,220.00
27. Aerator, piping, etc. in settling, mixing basins—lump sum	5,000.00	4,676.00	4,625.00
28. Alter exist'g. gate chamber and aerator—lump sum	700.00	1,735.00	1,675.00
29. Furnish and erect wash water tank—lump sum	3,600.00	3,921.00	4,150.00
30. Filtration plant superstructure—lump sum	63,500.00	48,886.00	55,700.00

II—Transmission Main—Section A	D	E	F
1. Cast iron pipe, 4 in.—321 l.f.	$0.493	$0.51	$0.537
2. Cast iron pipe, 6 in.—1921 l.f.	0.713	0.74	0.774
3. Cast iron pipe, 8 in.—481 l.f.	1.017	1.05	1.104
4. Cast iron pipe, 10 in.—161 l.f.	1.366	1.41	1.483
5. Cast iron pipe, 12 in.—161 l.f.	1.758	1.81	1.908
6. Cast iron pipe, 16 in.—3,248 l.f.	2.670	2.75	2.906
7. Cast iron pipe, 20 in.—161 l.f.	3.698	3.81	4.068
8. Cast iron pipe, 24 in.—6,096 l.f.	5.033	5.18	5.424
9. Standard fittings—24 tons	2,616.00	2,901.68	2,664.31
10. Special fittings—2 tons	298.00	288.80	233.87

III—Transmission Main—Section B	G	H	J
1. Hub end valves, 6 in.—9	$19.32	$19.32	$19.32
2. Hub end valves, 10 in.—1	46.00	46.00	46.00
3. Hub end valves, 12 in.—1	57.96	57.96	57.96
4. Hub end valves, 16 in. geared—4	133.01	133.01	133.01
5. Hub end valves, 20 in. geared by-pass—1	246.28	246.28	246.28
6. Hub end valves, 24 in. geared by-pass—4	354.38	354.38	354.38
7. Flanged end valves, 12 in.—1	62.14	62.14	62.14
8. Flanged end valves, geared, 16 in.—2	141.63	141.63	141.63
9. Cut in tees, 10 x 8 in.—3	61.00	49.40	63.02
10. Cut in tees, 4 x 4 in.—2	22.75	19.30	23.00
11. Cut in tees, 16 x 16 in.—1	212.21	169.50	201.10
12. Valve Boxes—22	3.00	3.00	3.00
13. Hydrants, 5 in.—3	61.44	69.00	61.44

IV—Miscellaneous	K	L	M
1. Yarn (free from oil)—1,200 lb.	$0.07	$0.08	$0.075
2. Lead—40,000 lb.	0.0375	0.0410	0.0459

V—Distribution Reservoir	N	P	R
1. Earth excavation & embankment—2,000 cu. yd.	$0.50	$1.00	$0.34
2. Borrow pit excavation & embankment—10,000 cu. yd.	0.50	0.50	0.34
3. Rock excavation & rockfill—9,000 cu. yd.	1.75	1.50	1.68
4. Concrete masonry, Class A—3,500 cu. yd.	19.50	20.00	23.50
5. Concrete masonry, Class B—30 cu. yd.	18.00	15.00	29.35
6. Portland cement, added or omitted—10 bbl.	2.40	3.00	2.70
7. Steel reinforcement, added, omitted—1,000 lb.	.04	.05	.04
8. Seeding—2 acres	450.00	200.00	73.10
9. Furnish, place, crushed stone, screened gravel—175 cu. yd.	2.00	2.00	3.58
10. Stone paving for ditches—230 sq. yd.	1.00	1.00	1.15
11. Furnish, install inside piping, valves, etc.—lump sum	5,300.00	6,000.00	5,410.00
12. Furnish, lay outside C.I. piping, 6 in.—60 l.f.	1.20	1.50	1.42
13. Furnish, lay outside C.I. piping, 8 in.—180 l.f.	1.65	2.00	2.09
14. Furnish, lay outside C.I. piping, 12 in.—325 l.f.	3.15	3.00	3.13
15. Furnish, lay outside C.I. piping, 16 in.—360 l.f.	4.75	4.50	5.10
16. Furnish, lay outside C.I. piping, 24 in.—50 l.f.	7.75	6.75	7.60
17. Furnish, lay open joint vitrif. pipe, 6 in.—1,500 l.f.	.30	.30	.29
18. Furnish, install m.h. frames, covers, vents, C.I. steps, etc.—lump sum	900.00	600.00	1,200.00

Fort Peck Dam Diversion Tunnels

FOUR concrete lined tunnels with inlet and outlet structures, averaging 28 ft. 8 in. in diameter, 27 in. in thickness and 6,160 ft. in length are under construction at Ft. Peck dam under contract awarded April, 1934, jointly to Silas Mason Co. and Walsh Constr. Co. Four of the eight 242-ft. vertical concrete-lined shafts located 2,400 ft. downstream from the upper portal will be circular in cross section with inside diameters of 60 ft. The other four shafts will be rectangular with inside cross sections 33x18 ft. Each shaft will rise 50 ft. above ground and will be lined with concrete averaging 5 ft. in thickness. The government furnishes cement, sand and gravel, liquid sealing solution and bituminous spray waterproofing. All other materials including reinforcing steel will be furnished by the contractors. The project financed through PWA funds calls for completion in 578 calendar days with minimum wages prevailing of $1.20 for skilled and 50c. for unskilled labor. Unit prices and totals for the three low bids are presented for comparison. (A) Silas Mason Co., New York, and Walsh Constr. Co., Davenport, Iowa, (contract) $7,311,940; (B) Frazier Davis Const. Co. & G. L. Tarlton, St. Louis, $7,485,536; (C) Missouri River Tunnel Const. Co., Kansas City, $8,202,380.

Note—Although this contract was awarded a year ago, the unit prices are given here to complete the record of Fort Peck contracts. Unit prices for the spillway gate structures, awarded in April, 1935, and other parts of the project will be published in subsequent issues.

—*Editor.*

FT. PECK DAM DIVERSION TUNNELS

	Bidder A	Bidder B	Bidder C
1. Open cut shale excavation—718,000 cu. yd.	$0.75	$1.25	$1.25
2. Open cut overburden excavation—308,000 cu. yd.	0.75	0.70	0.60
3. Pilot tunnel excavation—24,600 lin. ft.	45.00	60.00	40.00
4. Main tunnel excavation—709,200 cu. yd.	2.70	2.48	3.00
5. Shaft excavation—144,000 cu. yd.	6.00	2.80	3.00
6. Excavate for cutoff collars—8,100 cu. yd.	10.00	3.00	3.50
7. Furnish and place steel supports for main tunnels—3,300,000 lb.	0.05	0.10	0.06
8. Furnish and place steel supports for shafts—1,200,000 lb.	0.06	0.05	0.07
9. Furnish and place timber bents for pilot tunnels—35,000 lin. ft.	0.40	0.55	0.50
10. Furnish and place timber lagging for pilot tunnels—350 M. f. b. m.	50.00	50.00	50.00
11. Furnish, place sheathing, bracing for open cut excavation—280 M. f. b. m.	70.00	40.00	60.00
12. Place concrete in tunnel lining—235,800 cu. yd.	3.00	3.85	4.50
13. Place concrete in gate shafts—62,000 cu. yd.	2.50	2.55	4.70
14. Place concrete in cut-off collars—8,100 cu. yd.	5.00	4.00	4.50
15. Place concrete in Intake Structure—16,100 cu. yd.	5.00	3.40	7.00
16. Place concrete in downstream portals and retaining walls for outlet works—38,100 cu. yd.	2.50	2.85	4.50
17. Place concrete in approach channel lining—84,400 cu. yd.	1.25	1.20	3.50
18. Gunite coating—500 squares	15.00	5.00	8.20
19. Furnish and place struct. steel for reinforcing Intake Structure and forming rack guides—1,392,000 lb.	0.05	0.05	0.07
20. Furnish and place reinforcing steel—6,454,000 lb.	0.05	.035	.05
21. Furnish and place plate steel lining for shafts and shaft foundations—9,886,000 lb.	0.05	.045	.065
22. Apply liquid sealing solution—30,000 squares	2.50	1.80	1.50
23. Apply Bituminous spray waterproofing—5,000 squares	2.50	1.80	1.75
24. Drill grout holes—3,000 lin. ft.	0.50	1.00	1.00
25. Furnish, install grout pipes, connections—10,000 lb.	0.15	0.50	0.25
26. Grouting—50,000 cu. ft.	0.80	1.00	0.70
27. Furnish and place steel stripe—600,000 lb.	0.06	0.04	0.07
28. Furnish and place bronze piping—85,500 lb.	0.50	0.25	0.50

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